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HIGH PRESSURE TRIAXIAL COMPRESSION TESTING OF SASKATCHEWAN SILT AND BEARPAW SHALE

by

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The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance, a thesis entitled HIGH PRESSURE TRIAXIAL COMPRESSION TESTING OF SASKATCHEWAN SILT AND BEARPAW SHALE submitted by Garry Wood Hollingshead in partial fulfilment of the requirements for the degree of Master of Science.

ABSTRACT

Instability in the Upper Cretaceous clay shales of Western Canada has caused numerous slope failures. Effective stress analyses of the failures give factors of safety in excess of unity unless modifications are made to the failure criterion or to the measured shearing resistance of the material. Field evidence suggests that the stability of the Bearpaw formation depends on many geologic factors, the effects of which are not completely understood.

Values of shearing resistance for the shale have been reported for confined pressures up to 200 pounds per square inch. The aim of this investigation was to extend the range of confining pressure to 2000 pounds per square inch with the expectation that the resulting shearing resistance would be more indicative of field conditions.

Tests were conducted to determine the rate of membrane leakage at high cell pressures. Significant leakage was observed and is thought to occur primarily past the O-Ring bindings. A new rubber jacket for shale specimens is proposed.

Remolded Saskatchewan Silt was tested at cell pressures up to 1500 pounds per square inch to develop techniques for using the high pressure equipment and pressure transducers. These apparatuses performed satisfactorily. The silt exhibited unusual stress-deformation characteristics but the Mohr Rupture Line appears to be linear over this pressure range and gives effective cohesion of zero pounds per square inch and an angle of effective shearing resistance of 34 degrees.

Undisturbed Bearpaw Shale was tested at cell pressures up to 1000 pounds per square inch but membrane leakage prevented the successful conclusion of the tests at the higher pressures. The stress-strain curves peak sharply at one to three per cent axial strain and exhibit well defined residual strengths. Because of the limited number of completed tests and the influences of membrane leakage and rate of strain on recorded pore pressures, the reported values of shearing resistance are not considered reliable.

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GLOSSARY OF TERMS AND SYMBOLS

Terms

Angle of Shearing Resistance (Angle of Internal Friction), ϕ', ϕ -

Angle between the abscissa and the tangent of the curve representing the relationship of shearing resistance to normal stress acting within a soil.

Bulk Modulus - The pressure required to compress a liquid to zero volume.

Coefficient of Diffusion, D - The coefficient of proportionality in Fick's law of diffusion which states that the amount of material diffusing across a unit area per unit time is proportional to the concentration gradient.

Compressibility of Pore Fluid, C_v - The unit decrease in volume of the fluid per increment of pressure.

Compressibility of Soil Structure, C_c - The unit decrease in volume of the soil structure per increment of pressure.

Deviator Stress, $\sigma_1 - \sigma_3$ - The difference between the major and minor principal stresses in a triaxial test.

Initial Tangent Modulus - The slope of the stress-strain curve at the origin.

Jubilee Clips - adjustable circular steel clamps.

Milliequivalent per 100 grams of air-dried soil, me/100 gm ads -

The unit of cation exchange capacity; defined as one milligram of hydrogen or the amount of any other cation that will displace it.

Norton Tube - Trade name for a porous annular tube.

Overconsolidated Soil - A soil that has been subjected to an effective pressure greater than the existing overburden pressure.

Pore Pressure Coefficient, $\bar{A}$ - Ratio of change in pore pressure to change in deviator stress at any time during the shearing phase of an undrained test. Coefficient denoted $\bar{A}_f$ at failure.

Pore Pressure Coefficient, B - Ratio of change in pore pressure to instantaneous change in cell pressure.

Residual Strength - The ultimate strength exhibited by the soil at strains beyond that associated with peak strength.

Volume Factor (Compliance, Flexibility) - The volume of flow required to cause a pressure measuring instrument to record a pressure increase of one pound per square inch.

Symbols

a_v - coefficent of compressibility

c, c' - cohesion, effective cohesion

c_v - coefficient of consolidation

E - modulus of elasticity

k - coefficient of permeability

L.V.D.T. - Linearly Variable Differential Transformer

M_v - coefficient of volume compressibility

P.F.R.A. - Prairie Farm Rehabilitation Administration Soil Mechanics Branch

s - Shear Strength

u - pore water pressure

CHAPTER I

INTRODUCTION

1.1 Nature of the Problem

Vast areas of Western Canada and the North Central United States are underlain at shallow depth by Upper Cretaceous formations which are predominantly highly over-consolidated clay shales (Hardy, 1963). While these formations are apparently competent in their unweathered state and are referred to as bedrock materials geologically, they degenerate upon weathering to a much weaker highly plastic clay. The Bearpaw Shale of South Central Saskatchewan and Southern Alberta is one such formation.

The shear strength of clay soils is a complex physical property. Despite the fact that much remains unknown of the fundamental behavior of cohesive soils, research over the past thirty years has developed the triaxial compression test as a highly refined and useful method of investigating their shear strength. At least it has proven to be a

reasonable method of determining the strength parameters of normally consolidated and slightly overconsolidated cohesive soils for use in the Mohr-Coulomb theory of failure. At the same time, the results of standard triaxial compression tests do not give an acceptable solution to stability problems in overconsolidated clay shale (Peterson, 1960; Hardy, 1963). The behavior of geologically overconsolidated cohesive soil is far less tractable simply by virtue of the effects of its unknown stress history.

Slope stability in clay shales is of economic importance in Western Canda. It influences the location and design of many engineering works such as highways, bridges, petroleum pipeline river crossings and large dams. Although experience with the Cretaceous shales of Western Canada is not extensively reported, there are a number of well documented histories of slope failure which suggest that the normal engineering approach and stability analyses yield factors of safety in excess of unity when in fact failure has occurred. The original Peace River Bridge at Taylor, B.C. collapsed as a result of instability in the shale foundation of the north anchor block (Hardy, 1963). Consolidated undrained tests

with pore pressure measurements yielded values of shearing resistance considerably greater than the calculated stresses mobilized at the time of failure. The analyses of slides at Dunvegan and the Little Smoky River Crossing in Northern Alberta (Hardy, Brooker and Curtis, 1962) showed that pore pressures substantially greater than the measured field values were required to give factors of safety of unity. Research by the Prairie Farm Rehabilitation Administration Soil Mechanics Branch suggests values of ϕ' for Bearpaw Shale in an undisturbed condition in the order of 30 degrees. Yet natural slopes in this material in the area of the South Saskatchewan River Dam are as shallow as 12 to 1 (Peterson et al, 1960). These slope angles are much less than would be predicted on the basis of present laboratory tests.

It therefore appears justifiable at this point to consider the problem facing practising engineers who must deal with slopes in overconsolidated clay shales during the course of an engineering investigation and design. That failures occur is an irrevocable fact. A factor of safety of unity is implicit. If then, the generally accepted failure theories combined with the best analyses and strength

parameters available do not yield a factor of safety of unity, one is faced with the following alternatives. Either the theory or analysis must be modified or strength parameters must be obtained which, when applied to the theory, will in fact yield factors of safety of unity. There are in existence hypotheses which attempt both of these approaches (Hardy, 1964; Skempton, 1964) with varying degrees of uncertainty.

1.2 Purpose and Scope of the Investigation

In light of the foregoing, the immediate aim of this investigation was to extend the application of the triaxial compression test with the hope that it would yield more comprehensive knowledge of the shearing resistance of over-consolidated clay shale. Although this aim was not achieved, it is considered that a significant step in this direction has been made.

The work described in this thesis forms part of an intensive cooperative program undertaken by the Geological Survey of Canada and the Department of Civil Engineering, University of Alberta, aimed at solving the problem outlined in section 1.1 above. It was considered desirable to have

the capacity of testing the shale at any cell pressure up to that which represents the estimated preconsolidation pressure of the actual formation. Thus it was necessary to procure triaxial compression equipment capable of working pressures of 2000 pounds per square inch. This equipment was first used during the course of this investigation and is described in Chapter IV.

Intact Bearpaw Shale is an extremely impermeable material. Although no published laboratory data on the coefficient of permeability are available, it is believed a reasonable range would be 10^{-10} to 10^{-14} centimeters per second. Obviously very long duration tests are required to permit accurate measurements of the induced pore pressure. As an integral part of this research an attempt was made to determine a rate of strain suitable for measuring accurate pore pressures at the base of the specimen. To facilitate long duration testing three electrical pressure transducers were incorporated into the program. A description of these devices appears in Chapter IV.

In view of the long term high pressure tests envisaged, a program to determine the adequacy of rubber

membranes and O-Ring seals was initiated. In fact it was the inability to reduce membrane leakage to a tolerable magnitude which prevented the writer from achieving the original objective. As a result of this program the design of a new rubber jacket for use on shale specimens is proposed.

A series of consolidated-undrained compression tests with pore pressure measurements was conducted on remolded specimens of Saskatchewan Silt in order to develop techniques for using the new high pressure equipment and pressure transducers. With reasonable precautions this apparatus proved to be simple to use and capable of producing consistent results.

A review of the literature relevant to analyses of slope stability in clay shale formations is given in Chapter II. This review includes case histories illustrating the various approaches which have been used. Also included in Chapter II is a review of pertinent experience with high pressure testing. A fairly detailed description of the Bearpaw Shale formation "in situ" is given in Chapter III. Chapter IV is devoted to the experimental apparatus and Chapter V to the test procedures adopted. The experimental results are presented and discussed in Chapter VI and the conclusions and recommendations in Chapter VII.

CHAPTER II

LITERATURE REVIEW

2.1 General

In 1936 Terzaghi classified clays into three distinct categories; namely, soft intact clays, stiff intact clays, and stiff fissured clays. Subsequent research has developed our knowledge to the point where the shearing resistance of the intact clays, with the exception of extra-sensitive soils, is reasonably well understood. Although some aspects of the fundamental behavior of these soils are not yet clear, the results of laboratory strength tests have proven reliable in the analyses of actual problems. As pointed out in Chapter I this cannot be said of the highly overconsolidated clays and clay shales which fall into the third category. After extensive studies of the Bearpaw Shale at the South Saskatchewan River Dam site Ringheim (1964) concludes, "Field experiences have confirmed preconstruction indications that present laboratory testing methods are not

satisfactory for predicting the stability or shear strength of the soft disturbed shale." .

This chapter presents briefly a review of literature pertinent to the strength of stiff fissured clays and clay shales illustrating by means of case histories how the problem of slope analysis has been approached since the 1940's. The various failure or shear strength theories which are now available are mentioned and modern approaches to fuse the shear strength data and the failure theory in order to arrive at a reasonable factor of safety are illustrated.

In addition, this chapter outlines the progress made in certain aspects of triaxial compression testing and in the testing of rubber membranes for leakage.

2.2 Failure Theories for Saturated Cohesive Soils

The oldest and most widely used failure criterion is that due to Coulomb:

$$s = c + \sigma \tan \phi \qquad (1)$$

where s = shear strength

c = cohesion

σ = normal stress on the failure plane

ϕ = angle of internal friction.

Application of this criterion may be broadened by modification for Terzaghi's (1943) concept of effective stresses whereupon it becomes, for saturated cohesive soils:

$$s = c' + (\sigma - u) \tan \phi' \qquad (2)$$

where c' = effective cohesion

u = porewater pressure

ϕ' = effective angle of internal friction.

Equation 2 is normally represented graphically by the Mohr Rupture Line (FIGURE II.1) which represents the state prevalent in a condition of plastic equilibrium. This criterion is subject to many assumptions (Terzaghi, 1943) and influencing factors (Taylor, 1948; Whitman, 1960). It tacitly assumes that plastic flow, involving continuous deformation at constant stress, has no influence on values of c' and ϕ'. That is to say, a saturated cohesive soil has two distinctly separate and constant strength parameters.

A further elaboration is represented by the Krey-Tiedeman failure criterion (Tschebotarioff, 1951) which may be expressed as:

$$s = p_c \tan \phi_c + \sigma \tan \phi \qquad (3)$$

where $c' = p_c \tan \emptyset_c$

p_c = the preconsolidation pressure.

Originally proposed in terms of total stresses this criterion assumes that the branches CD and EF (FIGURE II.1) are always at an angle $\emptyset$ to the horizontal (Tschebotarioff, 1951). It is often regarded as being an unsatisfactory equation (Osterman, 1960). Nevertheless it does at least imply that the so-called cohesion is a function of the preconsolidation load and is not therefore a unique strength parameter for any given soil. The Coulomb-Hvorslev criterion is another modification which implies that cohesion is a function of consolidation pressure or void ratio (Hvorslev, 1960). It is of the same general form as that due to Krey.

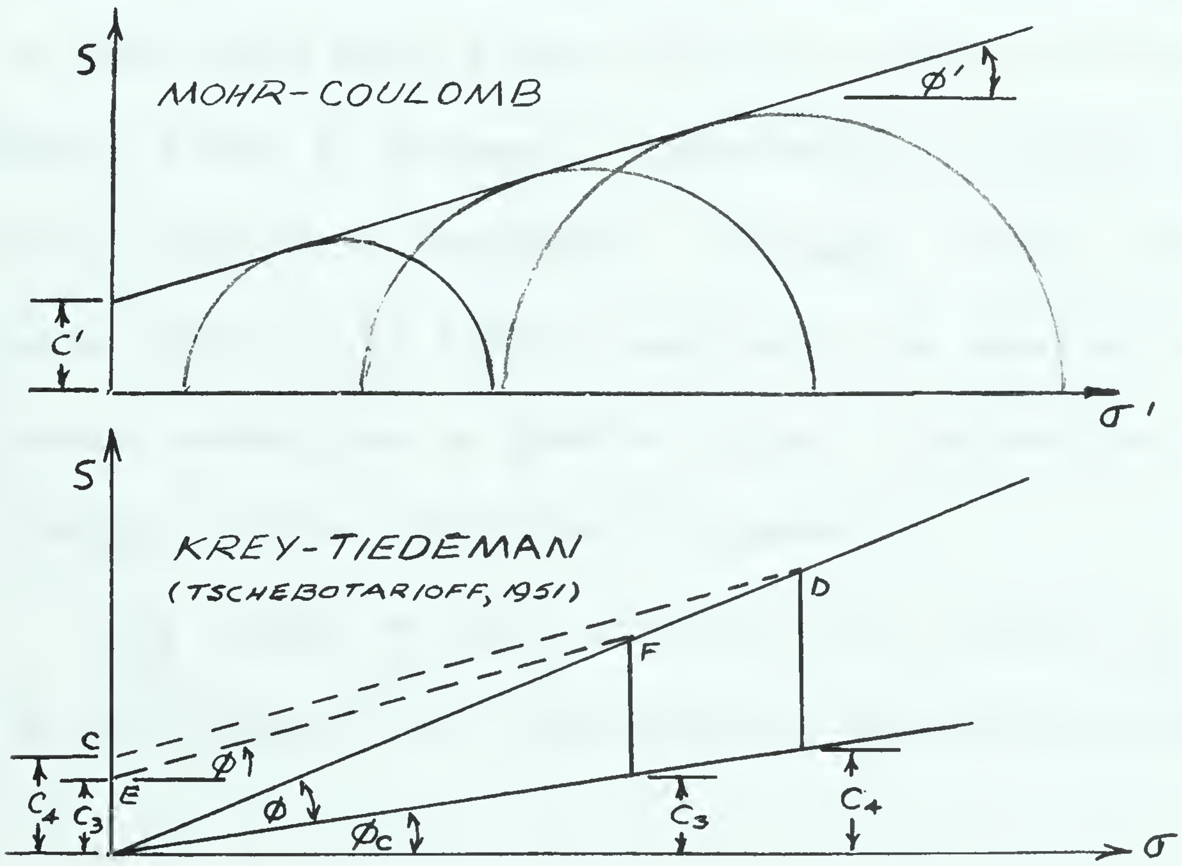

FIGURE II.1 Concepts of Shearing Strength

2.3 Mechanism of Slides in Fissured Clay

In 1948 Cassel presented certain histories of slope failures in fissured clay to the Second International Conference on Soil Mechanics and Foundation Engineering. He described these soils as deposits of much older geologic periods which have undergone great geologic changes involving consolidation under very heavy surcharges, lifting, denudation, folding by lateral pressures, and repetitions of these processes over long periods of time. He pointed out that they are not distinguished by a common mineralogical composition, origin, or method of deposition, identical grading, colour, plasticity or other index properties. Their common characteristic is the macroscopic structure which is disclosed when a lump of such clay at or below the plastic limit is dropped. Invariably it breaks up into small polyhedric fragments. Terzaghi (1936) recognized that these clays would likely represent the seat of very troublesome landslides on gentle slopes irrespective of the high strength of the individual fragments.

The origin of this structure is a result of variations in the vertical and horizontal pressures accompanying

geological movements and resulting shear stresses (Cassel, 1948; Brooker, 1964) and it is primarily because of the fissures which run at random through the mass of clay that it behaves differently from the essentially isotropic intact mass for which the failure theories prevail. As the stresses are relieved by excavation or erosion the fissures tend to open permitting access to surface and ground water. The mechanism of swelling and softening along open fissures results in a progressive reduction of strength.

2.4 Early Approaches to Stability Analyses

(a) Imperial College Approach

Cassel (1948) analysed several slips in excavated slopes on the basis of total stresses. These analyses indicated that the mobilized shear resistance along the failure surface was only one-fifth to one twenty-sixth of the undisturbed compressive strength of the material. He concluded that undrained compression tests on undisturbed samples did not provide sufficient information for estimating the factor of safety in slopes of fissured clay.

Skempton (1948) also described a number of failures in London Clay which, in every case, illustrated that the

undisturbed undrained strength was several times greater than the calculated strength on the slip surface assuming Ø = 0. On the basis of the progressive softening along fissures Skempton suggested that twenty foot vertical slopes would be stable for a few weeks, 2:1 slopes for ten to twenty years, and 4:1 slopes for perhaps fifty years. Permanently stable slopes on the other hand would not exceed about 10 degrees. The failures described by Skempton were used to illustrate a relationship showing the gradual reduction of cohesion with time (FIGURE II. 2).

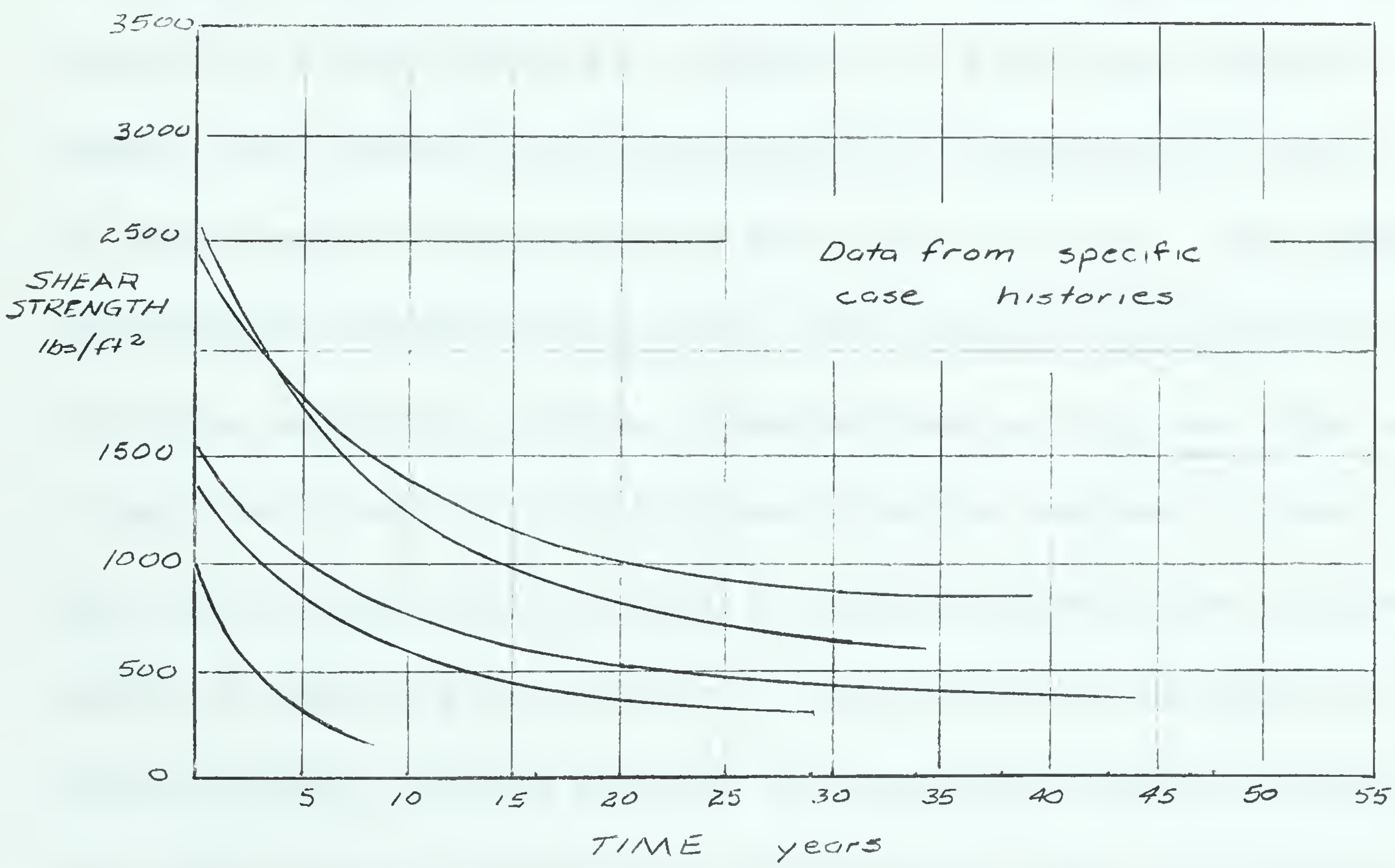

FIGURE II.2 Reduction of Strength with Time (after Skempton, 1948)

Further evidence suggesting that the maximum long term angle of slopes in London Clay is ten degrees was subsequently presented by Skempton and DeLory (1957). The effective strength parameters of London Clay were well established at this time and were reported as $c' = 250$ pounds per square foot and $\phi' = 20°$. The surveyed slopes were analysed as infinite slopes with shallow, plane failure surfaces. Reasonable agreement between observed conditions and laboratory results was obtained when the effective cohesion was assumed to be zero. Henkel (1957) described two long term failures in London Clay which indicated that one-half of the effective cohesion or about one hundred pounds per square foot was required if factors of safety of unity were to be obtained from the analyses. The general conclusions which may be drawn from these investigations of slope stability in the fissured London Clay are that on a long term basis the effective cohesion reduces to zero and the maximum stable angle is about one-half the drained angle of shearing resistance. The principle mechanism is stress relief through erosion or excavation which results in fissuring and subsequent softening of the soil adjacent to the fissures.

(b) American Approach

In contrast, a somewhat different approach has been used in stability studies of clay shale formations in North America. Binger (1948) presented the analytical treatment of two massive slides in the Cucuracha Formation of Panama which occurred during construction of the Panama Canal and which resulted in 50,000,000 cubic yards of additional excavation. These slides closed the canal to traffic for over seven months (Binger and Thomson, 1949).

The Cucuracha Formation comprises 40 to 60 per cent clay shale which is soft to medium hard, soapy, highly bentonitic, grey-green in colour and has many degrees of fracturing, jointing, and slickinsiding. Binger concluded from his studies that the shear strength of the Cucuracha Shale at failure had dropped, on the average, to 22 per cent of its original undisturbed value. He presents a slope chart intended for the design of future slopes in this material which is based on cohesion of 16 pounds per square inch and an angle of internal friction of 10 degrees (FIGURE II.3). These parameters were derived from analytical studies of a stable bank cut in the Cucuracha

Formation combined with results from simple laboratory tests designed to measure the friction which might be expected to develop along slickensides. The friction tests were made in a standard direct shear box on specimens cut from solid cores of sound clay shale, polished to simulate slickensides and brought into contact under water (Binger and Thomson, 1949). The resulting minimum angle of friction was 10 degrees. This method was adopted because it was considered that the actual strength of the formation could not be determined reliably by normal laboratory tests.

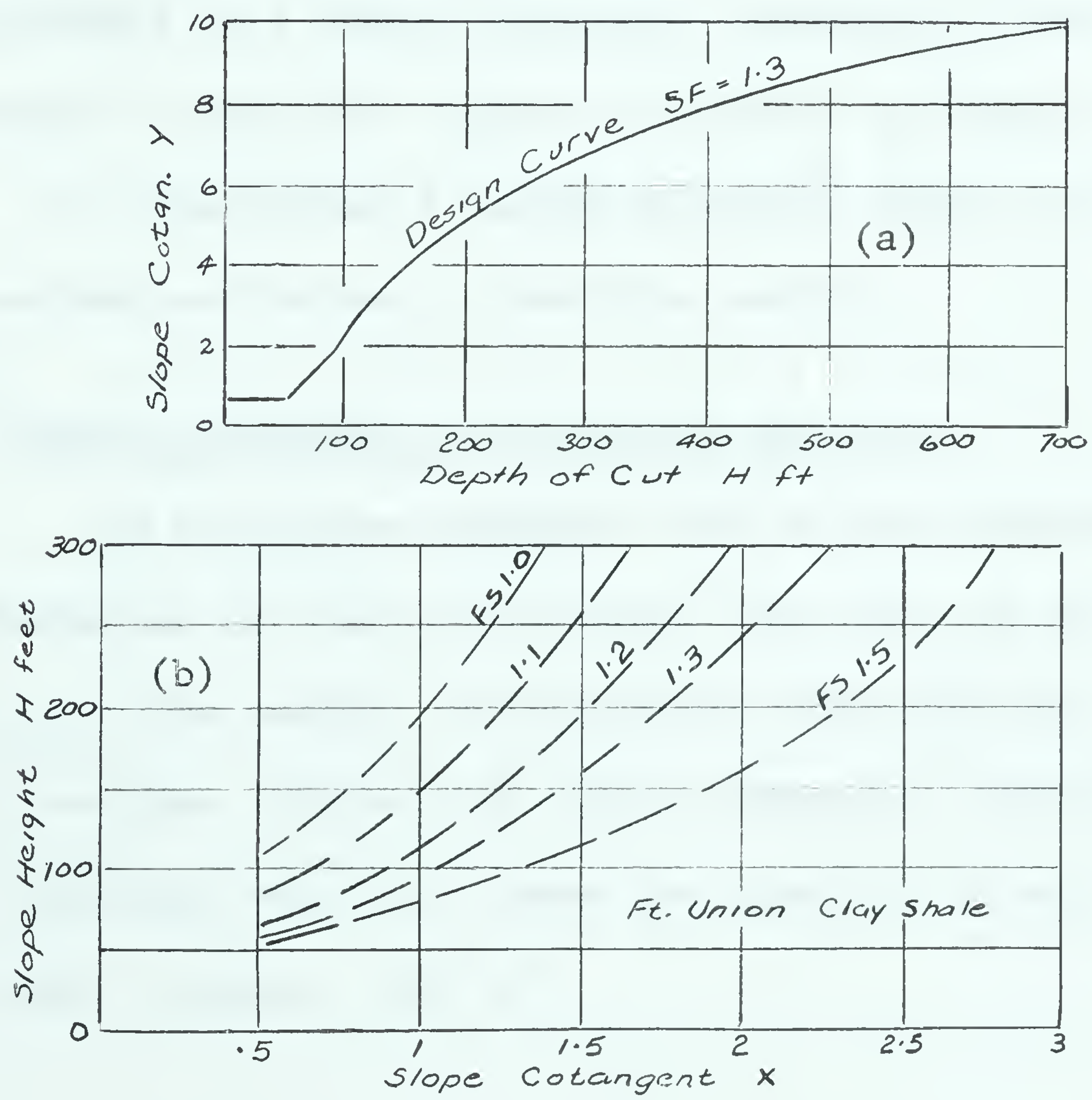

FIGURE II.3 Field Slope Charts
((a) after Binger, 1948;
(b) after Lane, 1961)

Lane (1961) extended the application of field slope charts to several clay shales in the North Central United States including the Bearpaw, Pierre, and Ft. Union formations. In Lane's chart (FIGURE II.3b) both stable and failed slopes are plotted. This method, in essence, makes use of nature's own "in situ" testing rather than the laboratory but is very much at the mercy of geological detail. In the opinion of Knight (1962) the slope chart is, at best, only an indication of the extremes to be considered because of the many unknown subsurface conditions which prevail in a shale formation. Generally, the method is limited to specific regions of similar geological conditions. It constitutes a design procedure based on field observations and is not an analytic method.

2.5 Modern Approaches to Stability Analyses

It has become apparent that in many instances the field behavior of overconsolidated clay shale is not in accord with the results of laboratory strength data. Some of the problems inherent in normal laboratory testing of these materials which may cause the apparent anomaly are as follows: (Olson, 1964 a)

(a) the clays are often fissured and therefore laboratory tests on small intact specimens do not apply to field conditions,

(b) difficulty of undisturbed sampling in hard formations,

(c) part of the strength may be of a viscous nature so that normal rates of testing overestimate field strength where rates of movement are far slower,

(d) release of lateral stresses and progressive softening in the field,

(e) swelling pressures and strength are influenced by electrolyte concentration in the porewater. Distilled water provides a different environment in the laboratory,

(f) accurate measurements of pore pressure in the laboratory are difficult because of the extremely low permeability.

The Prairie Farm Rehabilitation Administration have been concerned with the strength of the Bearpaw formation for over 10 years in connection with the design and construction

of the South Saskatchewan River Dam. They have conducted extensive triaxial compression programs and have concluded (Peterson, 1960) that the strength from drained tests on undisturbed or completely remolded specimens overestimates the stability of this formation. As a result they are forced to adopt an approach based on the study of natural slopes and slope failures to arrive at safe design slopes. Ringheim (1964) has analysed several movements in the shale at the dam site in terms of effective stresses with assumed pore pressures. Based on the results of this type of analysis the strength parameters selected for the soft shale for use in design stability analyses were $c' = 0$ p.s.i. and $\phi' = 9$ degrees. These are considerably less than the laboratory values reported by Peterson. High lateral pressures "in situ" have been measured by Peterson (1954) and estimated by Skempton (1961). Inclusion of these lateral forces in the analysis appears to have promise.

Investigators at Imperial College (Skempton and DeLory, 1958; Henkel, 1957) have arrived at reasonable predictions of stability in the London Clay by putting the effective cohesion equal to zero. This approach is justi-

fied by the apparent decrease of cohesion with time (Skempton, 1948). While good agreement is apparent for the London Clay it does not appear that this approach is sufficient for the clay shales of Western Canada.

A third approach is proposed by Hardy (1962) wherein the Mohr-Coulomb criterion is modified by the inclusion of a swelling pressure term;

that is

$$s = c' + (\sigma - u - p_s)\tan \phi' \qquad (4)$$

where p_s is the swelling pressure measured in the laboratory. In the case of the Dunvegan and Little Smoky River slides triaxial compression tests gave effective strength parameters of $c' = 0.2$ Kg./cm.^{2}, $\phi' = 21.5$ degrees; and $c' = 0.38$ Kg.cm.^{2}, $\phi' = 19.5$ degrees respectively. The average slope of the ground surface before sliding was 9 degrees at Dunvegan and 12 degrees at the Little Smoky and the effective stress analyses of both these cases resulted in factors of safety between 1.7 and 2.95. Most unrealistic pore pressures (up to 30 feet of water above the ground surface) were required to reduce the factors of safety to unity. The introduction of a swelling pressure term into the basic Mohr-Coulomb

equation, on the other hand, appeared to give more realistic assessments of the stabilities. This tacitly assumes that p_s is a neutral stress which has not been measured in the field.

The swelling pressure hypothesis brings into play the physico-chemical aspects of the behavior of cohesive soils. It is suggested (Hardy, 1964) that the swelling pressure is a manifestation of the net repulsive electrical forces between clay particles as visualized by Lambe (1960) and comprises both hydration and osmotic forces. It appears to be a significant property of the montmorillonitic clay shales of Western Canada. According to Hardy these pressures in the field must result in either expansion of the soil or reduction in effective stresses; in either case, a reduction in shearing strength of the soil will result.

For many of the investigations reported (see for example Hardy, Brooker and Curtis, 1962) open Casagrande standpipe piezometers have been used to obtain field pore pressures and then generally only following failure. It has been suggested (Terzaghi, 1943; Gould, 1949) that these piezometers will give reliable results only in fairly

pervious soils and are inadequate for obtaining porewater pressures in dense clays. As a consequence the reported results may be questionable largely because of inadequate piezometric installations.

Finally, it has been suggested that more realistic analyses of the long term stability of slopes in overconsolidated clays and clay shales may be made on the basis of residual strength (Skempton, 1964). FIGURE II.4a illustrates the stress strain curve obtained for an overconsolidated clay when sheared in a direct shear box under constant effective normal pressure and with full drainage allowed. If the test is continued to large strains beyond peak strength, the strength decreases to a limiting value called the residual strength. Both the peak and residual strengths, when plotted against effective normal pressure (FIGURE II.4b) appear to be in accord with Equation 2 except that the residual strength line shows c_r' to be very small or zero and ϕ_r' to be lower than the peak angle by one to ten degrees. Re-examination of some failures in natural slopes of London Clay noted earlier has shown that the "in situ" strength at the time of failure is often very close to the

residual strength. It is interesting to note that both Binger (1948) and Ringheim (1964) observed that the shale strength at the time of failure was probably a residual value.

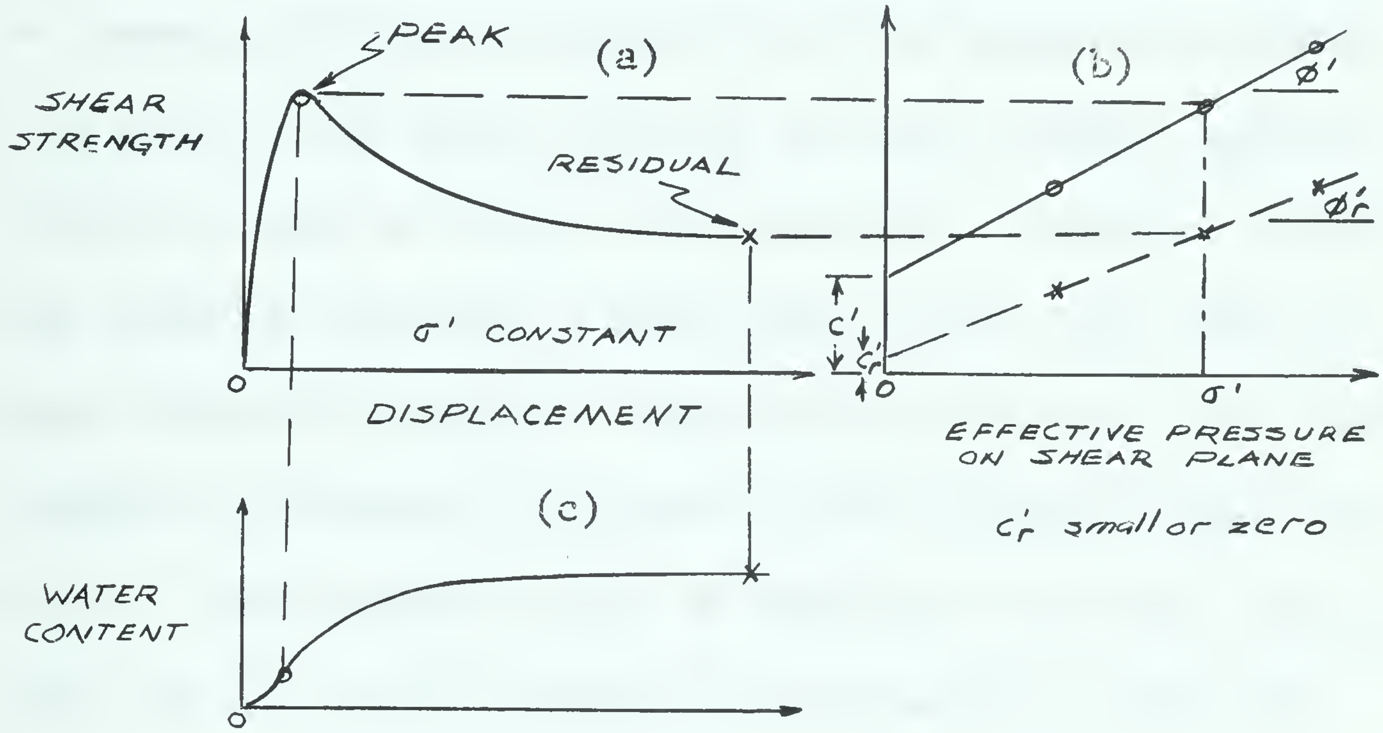

FIGURE II.4 Shear Characteristics of Over-Consolidated Clay (after Skempton 1964)

2.6 High Pressure Triaxial Compression Testing

Griggs (1936) has reported the results of early work carried out to study the strength behavior of rocks at high pressures. He constructed a steel "bomb" which was essentially a heavily armoured triaxial cell in which

kerosene was used for the cell fluid. Two pistons, acting together through a yoke, applied the deviator stress and controlled the chamber pressure simultaneously. Strength tests were conducted with this apparatus at cell pressures up to 10,000 atmospheres (150,000 pounds per square inch). At these confining pressures the rock apparently failed by plastic flow rather than by the more common rupture failures noted at lower cell pressures. Skempton (1960) has recently reviewed a great deal of the past work in high pressure testing of various materials and has illustrated clearly the downward curvature of Mohr Rupture Lines for porous media. The apparent angle of shearing resistance approaches that for the solid material as macroscopic voids are eliminated at high confining pressures.

Of more immediate interest are the apparatuses designed to test soils and soft rocks at high pressures. One of the earlier cells constructed for this purpose is described by Golder and Ackroyd (1954). The cell is constructed of steel, similar in design to modern triaxial cells, and can sustain working cell pressures of 1000 pounds per square inch. This pressure is applied hydraulically

by a motor-driven vane pump through an oil/water interchange. The cell is designed to test specimens ranging in size from 3/4 inches in diameter by 1½ inches in length to 4 inches in diameter by 8 inches in length.

The obvious practical application of triaxial compression testing at high confining pressures may be seen in the increasing number and height of earth and rock fill embankments being constructed today. For instance, the total pressure in the soil mass at the base of the Portage Mountain Dam in North Eastern British Columbia, which is approximately 650 feet high, would be in the order of 500 to 600 pounds per square inch. The high pressure cells in use today are intended primarily for testing the soils of which these high embankments and their foundations are composed. Development of the apparatus for both field laboratories and research in the United States has been reported by Warlam (1960, 1961). Various modifications of the Warlam triaxial cell (with aluminum chambers) are capable of withstanding laterial pressures of 1000 pounds per square inch. Piston friction is virtually eliminated by the use of ball bearing guides both within

and outside the chamber and the use of a flexible diaphragm around the piston to retain the pressure. In the Warlam design the pressure is supplied by compressed air.

By far the greatest majority of published tests results of high pressure work are for large diameter specimens of gravelly and sandy soils. Hall and Gordon (1963) have presented very interesting data on soils involved in the construction of the Oroville dam in which vertical pressures approach 40 tons per square foot. Their data (from tests on specimens up to 12 inches diameter at pressures up to 650 pounds per square inch) illustrate a definite reduction in the slope of the Mohr Rupture Line for the granular materials whereas the envelope for tests on specimens with a higher percentage (30 - 35%) of fines are very nearly linear. They attribute the reduction in ϕ' of the granular material largely to particle breakdown during straining at the higher confining pressures. Hirschfeld and Poulos (1963) have reported similar data for two soils at pressures up to 40 $Kg./cm.^2$. In this instance Mohr envelopes for both an undisturbed silt and a compacted sand exhibit significant decreases in the effective angle of

shearing resistance with increasing confining pressure. Thus the shearing strength of these materials obtained by extrapolating the envelope from 4 Kg./cm.^{2} to 40 Kg./cm.^{2} is overestimated by 35% in the case of the silt and 10% in the case of the sand. Again the reduction in strength at high pressures is attributed partly to particle breakdown.

2.7 Review of Membrane Testing

According to Poulos (1964) the first general interest in the effects of leakage on the results of triaxial compression tests was shown in 1939 by Mr. Henry Grace, a student at Harvard University. Since that time a number of investigators, principally at Harvard, have attempted to measure the flow of fluids through various types and combinations of rubber membranes.

Casagrande and Shannon (1948) report the results of a number of tests conducted on three different types of natural rubber membranes under a pressure differential of 6 Kg./cm.^{2} . They noted that gas often formed on the low pressure side of the membrane and suggested that it was either generated in the membrane or dissolved in the

liquid and liberated on the downstream side. Their results were inconclusive but appeared to show that the rubber was pervious to air, water, and castor oil in descending order. The continuing work at Harvard from 1949 to 1962, in which improved apparatus were used to test the permeability of different types of rubber to various fluids, has been reviewed and summarized by Poulos (1964). All of this work appears to have been concerned exclusively with flow through the membrane and did not consider such items as O-Ring seals as a significant source of leakage. Casagrande (1960) indicated that the problem of leakage had largely been eliminated for short term tests by the use of water in lieu of compressed air as the cell fluid. For long term tests he suggested leakage could be reduced by the use of double membranes soaked in silicone oil.

Seed (1960) has presented data which indicate that air diffused through two thin membranes separated by a layer of silicone grease at the rate of two cubic centimeters per day after an initial period of one and one-half days during which no significant leakage was observed. No leakage of water, even under a pressure differential

of 5 Kg./cm.^{2} was observed. These tests differed from the Harvard work in that the membranes surrounded a simulated specimen in the usual triaxial cell and thus end seals were not precluded. Leakage data for various thicknesses of membranes over periods up to 25 days under 30 pounds per square inch are illustrated by Akroyd (1957). These data were determined from tests conducted by the same method as that employed by Seed.

Rowe and Bardon (1964) have also reported the results of several interesting tests on saturated clay samples four inches in diameter and four inches high to investigate membrane leakage. Their data indicate that the rate of leakage was not significantly reduced by grease, silver paper, or the use of glycerine as the cell fluid. They concluded that much of the leakage occurred past the O-Ring seals and that it could be minimized by the use of polished end plates and Jubilee clips.

Poulos (1964) studied in detail the control of leakage in the triaxial compression test and concluded that, for twelve hour undrained tests on saturated specimens, leakage will not cause more than a two per cent reduction

in effective stress if the following are used:

(1) two 0.006 centimeter thick natural rubber membranes

(2) two O-Rings on both the cap and pedestal

(3) well polished cap and pedestal that are greased before application of the O-Rings.

For one hundred day undrained tests Poulos recommends the use of two 0.004 centimeter thick butyl rubber membranes. These membranes are apparently not yet available on a commercial basis.

2.8 Summary

This chapter has dealt with a review of case histories illustrating the apparent anomaly between the laboratory and field strengths of overconsolidated clays and clay shales. Four modern approaches toward solving or circumventing the problem were reviewed. These are:

(a) reduction of effective cohesion

(b) field slope charts

(c) swelling pressure hypothesis

(d) residual strength.

The progress made in high pressure triaxial compression testing and the testing of membranes for leakage

have also been reviewed. It would appear that a solution to the membrane problem is not yet available.

CHAPTER III

DESCRIPTION OF BEARPAW FORMATION

3.1 General

The Bearpaw formation and its stratigraphic equivalents, for example, the Pierre Shale, constitute the bedrock material of a band several hundred miles wide lying to the east of the Rocky Mountains and extending from Nebraska to the Yukon (Hardy, 1963; Ringheim, 1964). Their general extent in South Central Saskatchewan and Southern Alberta is outlined in FIGURE III.1 . All of these formations are of Cretaceous age and marine origin having been deposited in the Pierre Sea which extended from the Artic to the Gulf of Mexico. In their present form they are comprised of dense uniform grey clay shales with interbedded sandstone, siltstone, and bentonite layers. The predominant clay mineral present is montmorillonite. Upon weathering the shale may swell and soften, reverting to a highly plastic clay in most cases.

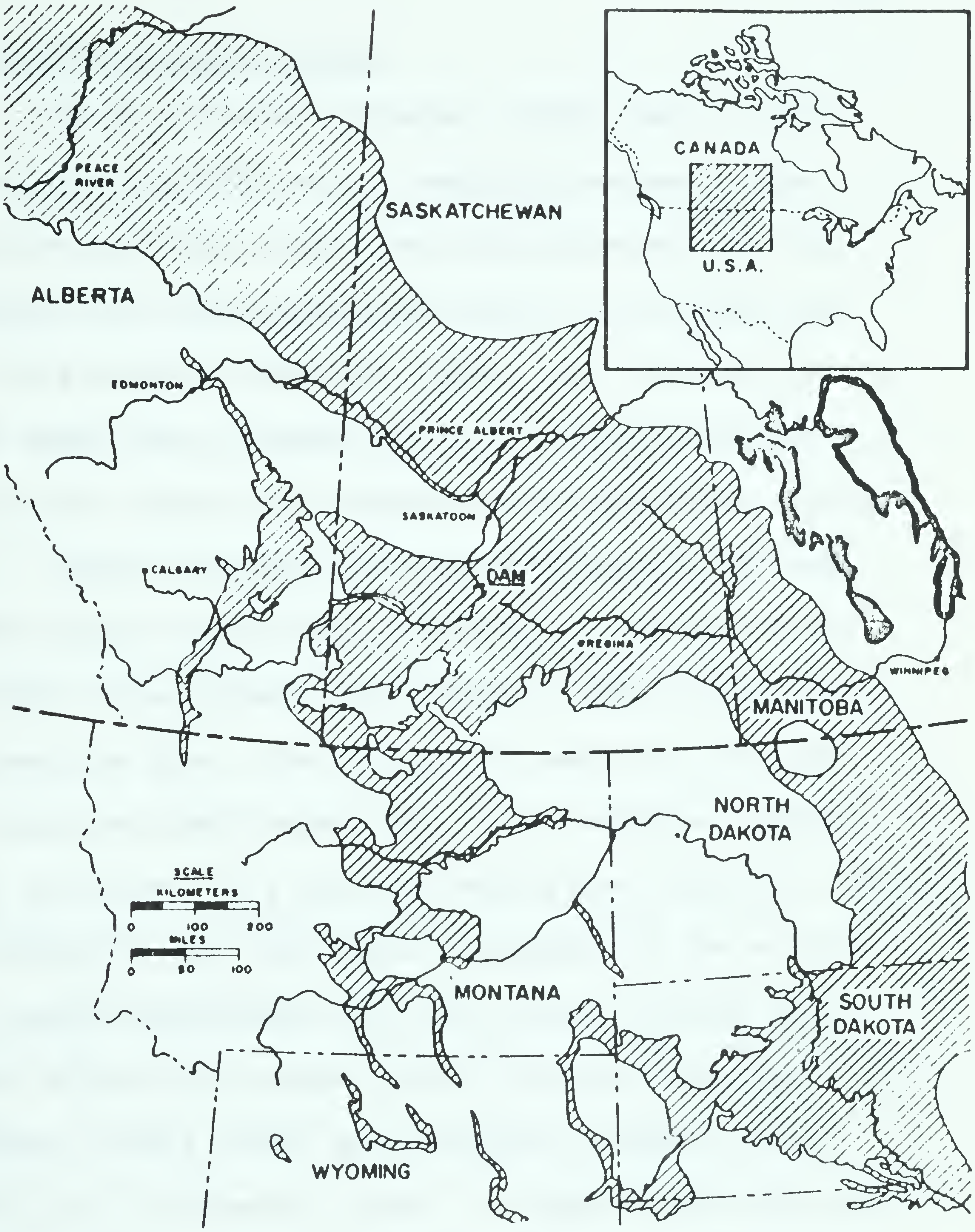

FIGURE III.1 Clay Shale Areas in Western Canada and United States (after Ringheim, 1964)

3.2 Bearpaw in Saskatchewan

It is estimated (Peterson, 1954) that at least 2000 and possibly 2500 feet of overlying sediments have eroded from the area of the South Saskatchewan River Dam. In addition the shales have been subject to at least one period of glaciation (Bayrock, 1965). The result is that in many areas these sediments are now overlain only by glacial drift except where they outcrop along river valleys.

Since deposition the Bearpaw formation has been uplifted several hundred feet evidenced by the fact that it outcrops at approximate elevations of 2000 to 3000 feet above mean sea level. As a result of overburden erosion it is also now slowly rebounding. Field evidence suggests that it is not yet in a state of equilibrium (Peterson, 1958). Observations of the river valley topography in the vicinity of the South Saskatchewan River Dam project provide ample evidence of ancient slumping and/or horizontal movement. The present valley slopes are apparently stable at about 8 : 1 to 12 : 1 (Peterson, 1954). A comprehensive treatment of the South Saskatchewan River Damsite geology is given by Pollock (1962).

The nature of the shale at this location has been studied for many years and is well documented in several publications by the P.F.R.A. (Peterson 1954, 1958, 1960, 1964). They have arbitrarily divided the formation into three zones, namely: soft, medium and hard shale from top to bottom, distinction between the zones being made on the basis of the range in moisture contents. In general the moisture content decreases and the spacing of fissures increases with depth. Thus the hard zone comprises shale with moisture contents below 27 per cent and joint spacing which generally exceeds 12 inches. This compares with moisture contents in excess of 31 per cent and joint spacing as small as 1/8 inch in the soft weathered surface zone.

3.3 Shale Outcrops Along the St. Mary River

In contrast with the above are the outcrops of Bearpaw Shale along the St. Mary River southwest of Lethbridge, Alberta. From an examination of the formation at this location it appears that the material is much less homogeneous. It is overlain by the St. Mary sandstone, Blood Reserve formation, lacustrine silts, and till, and underlain by coal seams and the Belly River formation. Slopes in the Bearpaw formation

of 41 degrees and greater to heights of approximately 200 feet above river level are prevalent in this area. There is some evidence of slumping but to a much lesser degree than that indicated along the South Saskatchewan River.

The weathered shale at the surface is highly fractured although the individual segments may be extremely hard. It is grey to brown in colour and contains numerous selenite crystals giving it a speckled appearance. Along many fractures it is highly oxidized causing it to be orange-brown in colour. There are several distinct, essentially horizontal bands of bentonite within the shale. These bands are approximately three to six inches thick, white to pale green in colour, and serve in some instances as good indicators of the dip of the strata and past movements. The bentonite also appears highly oxidized at the surface and contains occasional selenite crystals. Almost invariably the shale immediately beneath the bentonite is extremely hard and fractured whereas the shale immediately above the bentonite is of a relatively soft and plastic consistency grading upwards into the hard, fractured material. This suggests that, where intact, the bentonitic seams may control not only the movement of ground water through the

shale but also its stability. On most exposed slopes along the St. Mary River evidence of seepage is distinct and manifested by an accumulation of salts presumably leached from the soil above.

3.4 Red Deer River Valley

The Bearpaw formation also outcrops along the Red Deer River between East Coulee and Finnigan, southwest of Drumheller, Alberta. At East Coulee it is overlain by 100 to 200 feet of Edmonton formation sandstones and shales and at Finnigan by glacial drift only. While slump topography is not as spectacular as along the South Saskatchewan River there is some evidence of ancient slump activity. On the surface the shale is highly desiccated and resembles closely packed light brown popcorn balls. It is weathered and desiccated for some depth. The overall impression gained from the topography is that the nature of the Bearpaw formation at this location is between that prevalent along the South Saskatchewan River and that along the St. Mary River.

3.5 Geological Factors Influencing Stability

As a result of field studies it is now possible to describe a number of factors which appear to influence the

stability of the Bearpaw formation. These factors provide a rational basis for explaining the regional differences in behavior noted above. They are tentatively tabulated as follows (J.S. Scott, 1965):

(a) Geological history and morphology of the river valley,

(b) Lithology of the formation as a function of depositional environment with emphasis on the presence of minor geological details such as the presence of bentonite seams,

(c) Hydrogeology with reference to the geological factors affecting ground water discharge,

(d) Chemistry of ground water as related to the development of osmotic and rehydration pressures within the soil mass,

(e) The stress history of the soil mass in which the shale has developed as manifested in the overconsolidation ratio.

In addition to the above factors it should be noted that the occurrence of selenite ($C_a\ SO_4 \cdot 2H_2O$) suggests that calcium may be the predominant cation in the shale along the St. Mary River. The shale of South Central Saskatchewan on the other hand is known to have a high proportion of adsorbed

sodium (TABLE V.1). Since recent research at the University of Alberta has shown that the presence of sodium reduces strength, the apparent difference in physical chemistry of the soil-water system may be largely responsible for the greater stability of shales in the St. Mary River region.

3.6 Summary

It is evident that the shale properties are different at the three locations discussed in this chapter. Therefore, when one refers to Bearpaw Shale as a troublesome material which is stable only on very shallow slopes, one must take care to specify the area to which he refers. Obviously the shale is not always a source of trouble. It is the properties of the apparently unstable material along the South Saskatchewan River in Central Saskatchewan with which the remainder of this thesis is primarily concerned.

CHAPTER IV

EXPERIMENTAL APPARATUS

4.1 General

The nature of this program made it necessary to procure certain new apparatus, the essential components of which are a high pressure triaxial compression system and an electrical pressure transducer. It was considered that the most useful information of the stress strain characteristics of the Upper Cretaceous Shales will be obtained only if the program is extended into pressure ranges compatible with the geologic stress history. The preconsolidation load has been estimated at 100 to 150 tons per square foot (Peterson, 1954). On this basis 2000 pounds per square inch (144 tons per square foot) was selected as the maximum working pressure.

The electrical pressure transducer is a convenient means of monitoring pore pressures accurately over long periods of testing. These devices are also suitable for

integration into automatic data processing systems and have come into increasing use in soils testing in recent years. It also appeared desirable early in the program to design and construct a system for the express purpose of examining the permeability of rubber membranes at high pressures.

The high pressure system, transducers, and membrane test apparatus are now operative. They are described briefly in the following paragraphs and discussed more fully in Appendicies A, B, and C respectively.

4.2 High Pressure Equipment

This apparatus comprises a combined constant pressure and pore pressure system, two high pressure cells and auxiliary equipment. The pressure is generated hydraulically by weights suspended from a needle-like ram floating in an oil filled balancing cylinder and is transmitted to the cell through an oil/water reservoir. Five pounds produces approximately 100 pounds per square inch. Two motors provide for the continuous maintenance of pressures up to 2000 pounds per square inch over extended periods. One motor operates continuously to rotate a bushing on

the ram reducing friction. The second motor is activated automatically as the ram sinks in the cylinder. It drives a pump which forces oil into the cylinder, raising the ram and restoring the desired pressure. When the system is operating properly pressure can be maintained within $\pm$ 0.2 per cent. The reduction of pressure at the completion of a test is controlled by a needle valve. Good control is important to avoid damaging the end of the ram by contact with the bottom of the balancing cylinder.

The pore pressure measuring apparatus is a standard Bishop null indicator adapted for the higher pressures. It consists of a vertical capillary tube extending into a mercury-filled trough, a small bore control cylinder, a steel Bourdon tube pressure gauge, and a length of steel tubing. This apparatus was not employed except as a means of calibrating two of the transducers.

The cells are constructed of steel and brass except for three windows provided for viewing the specimens. They are equipped with removable 1.5 inch diameter pedestals

incorporating two drainage ports. Load is transmitted to the specimen by means of a one inch diameter steel piston shaped at the bottom end so that it seats directly in a conical recess in the loading cap. The pressure seal around the piston consists of a single O-Ring embedded in a closely machined, fixed bushing.

4.3 The Pressure Transducer

The usual laboratory methods of measuring pressure (mercury monometers and Bourdon gauges) cannot be used to measure pore pressures in a soil specimen because of the flow of water required to make them register. Thus there has been a continuing search in soils research for an instrument which is adequate for this purpose. A number of such instruments employing a null method of measuring pore water pressures have been devised (Andresen, 1957; Bishop and Henkel, 1962). These instruments suffer from the disadvantage that they must be continously nulled manually; a serious drawback for tests which may extend over many days or even weeks. This deficiency has been overcome by Raymond (1963) who developed an inexpensive self-adjusting indicator but only at the expense of a greatly increased

system flexibility.

The pressure transducer is a relatively simple device which converts pressure on a diaphragm into an electrical signal. It consists of a miniature unbonded strain gauge in the configuration of a four active-arm resistance Wheatstone bridge which senses the deflection of the diaphragm under pressure. The strain gauge is excited by a six volt AC or DC supply and outputs approximately 24 millivolts full scale. This ouput may be recorded by a millivoltmeter, strain indicator, oscilloscope, X - Y plotter or any other similar recorder. A battery powered SR4 strain indicator was found to be the most satisfactory available instrument and was used for all of the tests reported herein.

The most important feature of a good pore pressure instrument is a low volume factor or compliance. For the Dynisco PT25 transducers used in the investigation this factor is approximately 0.4×10^{-6} in.^{3}/p.s.i. (Richardson and Whitman, 1963) which compares very favourably with other systems (Bishop and Henkel, 1962). Thus the transducer combines the advantages of self-adjustment and low compliance.

It is, however, an expensive item.

In order that advantage may be taken of the low compliance, the transducer should be connected to the specimen by the shortest possible column of water. To achieve this end, a small stainless steel adaptor was designed and constructed to hold the PT25 transducer. This adaptor is compatible with both the low and high pressure cells and in operation it simply replaces one of the drainage port Klinger valves. The transducer is then threaded into the other end care being taken to ensure that the recommended mounting torque is not exceeded. To ensure that no air is present in the system, deaired water may be flushed through the pedestal, past the transducer diaphragm, and out through a miniature shut off valve in the top of the adaptor. With this valve closed the transducer is ready to record accurate pore pressures. While pore pressures were measured at the specimen base during this program, the same system could be adapted to the measurement of pore pressures at any other point in the specimen.

4.4 Membrane Test Apparatus

The test cell was designed to withstand 2000 pounds

per square inch working pressures and is constructed of steel throughout. The base plate is equipped with a 1.5 inch diameter pedestal on which simulated specimens are placed. The membranes and O-Ring seals can then be used in the usual manner and observations made of the leakage occurring under conditions identical to that found in a normal triaxial compression test. The simulated specimen is a three inch length of porous stone tube of 1.5 inches outer diameter. Leakage of cell water into the specimen is observed in a 25 millilitre stopcock burette attached to a drainage port in the baseplate and pedestal. Cell pressures are supplied by either the hydraulic system described in section 4.2 or compressed nitrogen.

CHAPTER V

SPECIMEN PREPARATION AND TEST PROCEDURES

5.1 General

The purpose of this chapter is to outline the methods employed in preparing silt and shale specimens for testing. In addition, the techniques developed for employing transducers and conducting high pressure triaxial compression tests are reviewed. Tests conducted concurrently to determine the leakage characteristics of various membrane and binding arrangements are also described herein. It must be borne in mind that much of this work was of a preliminary nature and while the methods adopted appeared to work satisfactorily there are no doubt areas where they can be refined still further.

5.2 Preparation of Silt Specimens

Saskatchewan Silt is a postglacial fluvial deposit found near the confluence of the North Saskatchewan River and Whitemud Creek at Edmonton. It has been used for routine

testing at the University of Alberta for a number of years. The Atterberg limits and other index properties are shown in TABLE V.1 .

Several hundred grams of air dried Saskatchewan Silt were sieved on a No. 40 mesh. The minus 40 fraction was thoroughly mixed with distilled water to a moisture content of approximately 30 per cent and allowed to soak over night. This moisture content is slightly above the liquid limit. The specimens were formed by consolidating the slurry one dimensionally in a series of lucite molds. These apparatus consist of two concentric lucite tubes 2.75 and 1.75 inches in diameter. One or two filter papers were wedged between the inner tube and bottom porous plate and the slurry was placed within this tube with the aid of a spatula. The tube was tapped sharply on the table to remove air bubbles after each lift was placed. When the slurry was within 0.5 inches of the top of the tube the loading cap with vertical drains was inserted and the whole placed within the outer lucite mold. The annular space between tubes was filled with distilled water and the slurry subjected to a vertical load by means of weight suspended from the end of a lever arm.

TABLE V.1

INDEX PROPERTIES OF TEST SOILS

PROPERTY	SASKATCHEWAN SILT	BEARPAW SHALE
Natural Water Content	-	23 - 27%
Specific Gravity	2.67	2.70
Atterberg Limits		
Liquid Limit	28.8	104.7
Plastic Limit	23.5	39.9
Plasticity Index	5.3	64.8
Shrinkage Limit	21.4	19.3
Grain Size Distribution		
% Sand Sizes	20	7
% Silt Sizes	70	36
% Clay Sizes	10	57
Activity	-	1.14
Mineralogical Composition[(a)]		
Kaolinite)	-	20%
Chlorite)	-	
Montmorillonite	-	80%
Cation Exchange Capacity		42.8me/100gms ads.[(b)]
Permeability cms/sec	10^{-7} @ e = 0.52[(c)]	$5x10^{-9}$ to $5x10^{-10}$ (d) $9x10^{-11}$ @ e = 0.72[(e)]

(a) Determined by Alberta Research Council on clay fraction
(b) 50% Sodium
(c) Extrapolated from higher void ratios
(d) From field permeability tests (Ringheim, 1964)
(e) Calculated from one dimensional consolidation

Vertical deflection dial readings were plotted against logarithm of time for each load increment. Specimens number 1 to number 10 inclusive were consolidated to an approximate vertical effective stress of 2.4 $Kg./cm.^2$. Specimens number 11 to 16 inclusive, which were prepared for testing at higher pressures, were consolidated to about 4 $Kg./cm.^2$. In all cases complete consolidation under the highest loading was achieved within two days. After consolidation the specimens were extruded, waxed and stored in the moist room until required for testing.

It should be noted that an earlier attempt had been made to form specimens of a very uniform silt from Johnson's Crossing, Mile 870 Alaska Highway, Yukon. These specimens lacked sufficient strength, after consolidation under 2.4 $Kg./cm.^2$, to retain their shape even with the most careful handling. They were therefore abandoned in favour of the Saskatchewan Silt. Although this latter material is stiffer than the Johnson's Crossing Silt, the remolded specimens must be handled with extreme care to avoid distorting or tearing them.

The specimens were trimmed with a wire saw while

being held in a trimming apparatus. Since the soil tears easily only very thin slices may be taken and to avoid tearing large chunks from the edges slicing must be conducted from both ends towards the center. Approximately 0.1 inches thickness was removed using the trimming edges set for a diameter of 4.04 centimeters. This material was discarded. The apparatus was then reversed and the specimen trimmed to size (1.4 inches diameter) in the same manner. These trimmings were retained for moisture content determination. Great care was necessary in trimming the ends to avoid distorting the specimen shape. A sawing action with very little pressure applied, combined with rotation of the specimen appeared to be satisfactory.

In order to prepare 1.5 inch diameter specimens for the high pressure apparatus, angle irons with a thickness of 0.051 inches were affixed to the 1.4 inch side of the trimming device. Exactly the same trimming procedure was followed. The resulting specimen diameter was about 3.86 centimeters or 1.52 inches.

5.3 Preparation of Shale Specimens

All of the shale tested during the course of this program was obtained from a single borehole at the site of the South Saskatchewan River Dam (Rivard, 1964). The samples were taken from a depth of 97 to 103 feet and are representative of the zone of hard shale. They arrived at the University of Alberta in the form of waxed six inch cores 12 to 18 inches in length.

The shale was very dense, dull grey in colour, and essentially homogeneous and intact. There were several small but very hard inclusions and lenses of siltstone and fine sandstone as well as the occasional tiny marine fossil dispersed throughout. The specimens were very susceptible to cracking and a number of fine cracks often developed during preparation. If the specimen was allowed to dry for a few minutes its fractured nature became very evident. The results of classification tests are shown in TABLE V.1 .

The cores were much too hard to penetrate effectively with the sharpest knife. After some experimenting a woodworking band saw was found to be the most expedient method

of reducing the samples to workable sized pieces. Initially therefore, a slice, 3.5 inches to four inches thick, was removed from the sample using the band saw. This slice was then sawed into sectors as required. Exposed surfaces of the unused portions were covered, rewaxed and these portions stored in the moist room for future use. The sample placed in the trimming device was then a pie-shaped piece (FIGURE V.1).

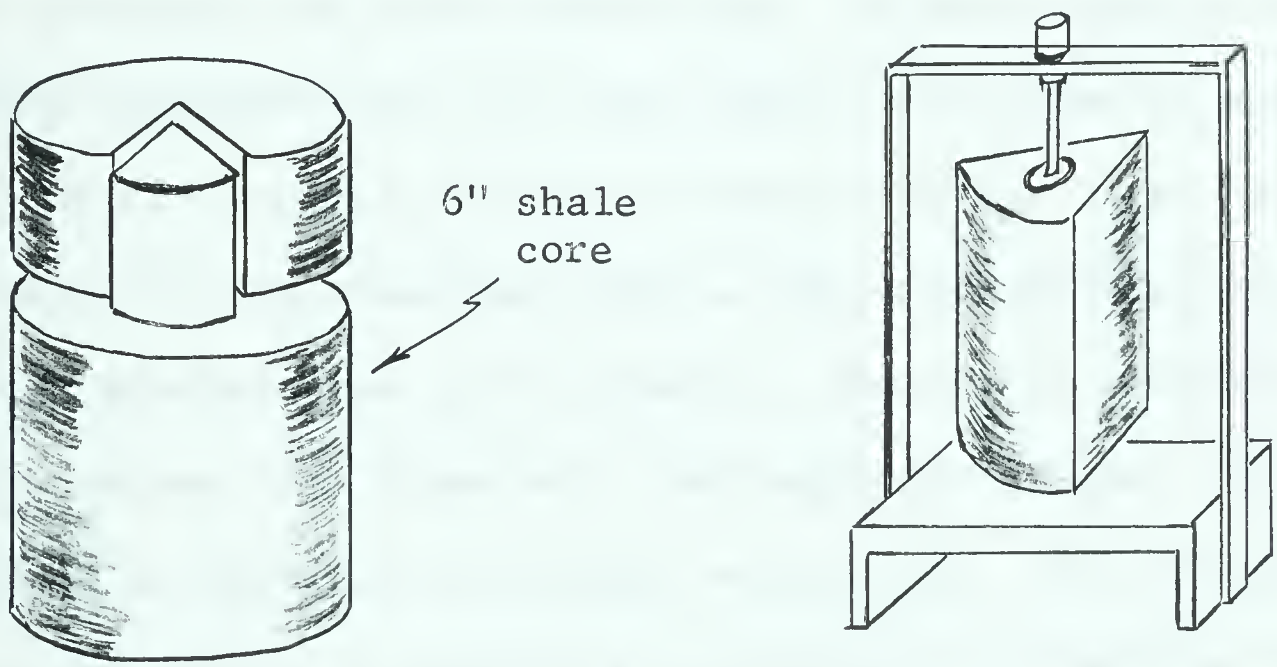

FIGURE V.1 Method of Carving Shale Specimens

A very sharp butcher knife with a reasonably straight blade for at least 6 inches appeared to be the most satisfactory instrument for carving the shale. After the corners and exposed faces were removed the trimmings were used for determining the initial moisture content. Additional trimmings

were saved from all specimens for performing the classification tests. Final trimming to size was done with a sharp plastic straight edge which, in effect, peeled or scraped off the excess material. Trimming of the ends was accomplished in the same manner; that is, with the knife first followed by the plastic straight edge.

Cracking often occurred in the specimens during the course of the above operations. In some cases it was hardly noticeable but in other cases it resulted in specimens which consisted of two or three separate pieces and which had to be abandoned just as they were nearing completion. If the specimen was still intact at the end of trimming it was measured (six diameters, two lengths), weighed, and mounted in the cell as quickly as possible. No difficulty was experienced in producing an excellent cylindrical specimen with squared, plane ends so long as they did not crack. The entire operation took about one to one and one-half hours.

5.4 Calibration of Pressure Transducers

Calibration data in the form of full range sensitivity in millivolts per volt excitation and per cent deviation from linearity at certain percentages of full range pressure were

provided by the manufacturer for each of the transducers. However, since a strain indicator was adopted as the best available recording instrument, this data proved to be of little direct value. Consequently each transducer was calibrated in the laboratory before use. The details of this procedure and resulting calibration curves are given in Appendix B and summarized in the following paragraphs.

For the 100 p.s.i.a. transducer, a factory calibrated Bourdon gauge of the same range was used. The 300 p.s.i.a. and 2000 p.s.i.a. transducers were calibrated against the pore pressure Bourdon gauge supplied with the high pressure apparatus. This gauge was first checked by means of a dead load tester and found to be slightly in error.

The transducers were adapted to the triaxial compression cells and pressure transmitted to the diaphragms through the cell water. Several readings on the strain indicator were averaged for each increment of pressure from atmospheric to full scale. The ratio of pressure to divisions (micro-inches per inch) was computed for each increment and plotted against the corresponding number of divisions above that representing atmospheric pressure. In

the cases of the 100 p.s.i.a. and 2000 p.s.i.a. transducers this resulted in a single curve but for the 300 p.s.i.a. instrument a significant hysteresis was apparent thus necessitating a two branch curve.

5.5 Mounting and Consolidating the Specimens

The first series of tests was conducted on 1.4 inch diameter silt specimens using the Bishop null indicator for measuring pore pressure (Bishop and Henkel, 1962). The pore pressure system was deaired as recommended by Bishop and Henkel until 100 pounds per square inch caused less than a 0.5 inch rise in the mercury level. The porous stone was boiled in water and water was flushed through the pedestal to ensure saturation. No. 54 Whatman filter paper was placed between the specimen and the porous disk; the whole placed on the pedestal and enclosed with a rubber membrane fixed with O-Rings. The cell was filled with deaired water topped with about 0.25 inches of oil. During consolidation water from the specimen was expelled into a 25 ml. stopcock burette. Following the consolidation phase the drainage line was removed and replaced by a back pressure line. To accomplish this operation the cell was set in a

pan of water so that at least two inches of water was above the drainage valve.

This series of tests was repeated using the pressure transducer in lieu of the null indicator. The procedure was similar except as follows. One of the Klinger valves associated with the pedestal drainage port was removed and replaced by the transducer adaptor. The transducer was screwed into the adaptor until finger tight. With the rubber dam extending above the surface of the pedestal, water was drawn from the burette, across the pedestal and out through the miniature shut off valve in the adaptor by means of slight suction applied to a line fitted to this valve. The rubber dam was then rolled back and the specimen slid on to the pedestal so as to preclude air from the system as far as possible. The rubber dam was then snapped up onto the porous stone and the membrane and O-Rings positioned to complete the mounting procedure. Following consolidation the cell was placed in a pan of water so that the base was submerged while the drainage line was replaced by the back pressure system.

Both cell and back pressures for all tests up to 120 pounds per square inch cell pressure were provided by self-compensating mercury pots similar to the system described by Bishop and Henkel (1962).

1.4 inch diameter specimens of shale were mounted and consolidated in precisely the same manner except that additional drainage aids in the form of a circular filter paper at the top of the specimen and slotted filter paper on the periphery of the specimen were used. The slotted filter paper was in contact with the porous disk to promote rapid drainage and pore pressure equalization. Several different combinations of rubber membranes, layers of silicone grease and rubber O-Rings were used in an attempt to reduce leakage of cell fluid into the specimen. The lengthy durations of consolidation and shearing phases combined with leakage of oil around the piston necessitated a greater thickness of oil seal.

The high pressure cell (section 4.2) is equipped with a 1.5 inch diameter pedestal. This necessitated the modification of the trimming device described earlier (section 5.2). The transducer was used in the same manner as with the low pressure cells except that it became necessary to wrench the transducers to eliminate leakage. A make-shift torque wrench consisting of a 0.75 inch open end wrench and fisherman's balance was used for this purpose.

It is most important that 100 inch-pounds mounting torque is not exceeded or distortion of the transducer body causing a permanent shift in the electrical output may result.

The pedestal was saturated and specimens mounted as previously described. The cell was filled with distilled deaired water through the cell pressure line by gravity feed. Although a thin film of oil was provided in the earlier tests, either by pouring it directly onto the base plate or through the oil injection valve, it was later found that this refinement was unnecessary. Once the cell was filled consolidation was conducted in the usual manner. Since the silt consolidated rapidly, the cell pressure was raised to the test value in increments with full consolidation permitted under each increment.

5.6 Testing the Specimens

Nineteen consolidated-undrained tests with pore pressure measurement were conducted. Of this total, eleven tests (two of which were later discarded) were run on remolded silt specimens, and eight on specimens of undisturbed shale. In addition to these several specimens of each material were set up in the triaxial cells but were

subsequently abandoned because of apparent membrane leakage, equipment difficulties or other problems. The tests reported may be divided into the following groups:

(a) Four silt specimens: tested to compare pore pressure measuring systems

(b) Five silt specimens: tested at high cell pressure to determine a Mohr Rupture Line

(c) Five shale specimens: tested to determine a suitable rate of strain

(d) Three shale specimens: tested to determine a Mohr Rupture Line.

Following consolidation as outlined in the previous section the specimens were generally subjected to pore pressure reaction tests. With the drainage valve closed the cell pressure was instantaneously increased by 10 to 100 pounds per square inch and the consequent increase in pore pressure observed with time. In some instances observations were discontinued after two minutes and in other cases they were extended for 40 minutes. The "unload" reactions were also observed. Invariably these reactions were poor without the benefit of back pressure.

After the back pressure line was secured to the cell, the cell pressure and pore pressure were increased simultaneously so as to maintain a constant value of σ_3'. Back pressures varied from 20 pounds per square inch to 50 pounds per square inch, the magnitude being governed by the pore pressure reaction and/or the capacity of the equipment. Periods varying from several hours to three days were allowed for equilibrium to be achieved and the reaction tests were then repeated.

Deviator loads were applied at a controlled rate of strain by means of a five ton Farnell loading machine and recorded with external proving rings of a suitable range. Previously calibrated proving rings with capacities of 400 pounds, 1500 pounds, and 4000 pounds were used and loads were corrected by subtracting the initial force on the piston due to the cell pressure.

Since the top of the specimen could not be observed in the high pressure cell it was necessary to rely on the proving ring dial gauge to ascertain when the piston was properly seated. As a guide, the force on the piston and corresponding dial reading were estimated before advancing

the piston. As a further check the total distance of advance was observed with the strain dial. This distance was about 0.5 inches.

Each silt specimen was strained at a nominal rate between 0.003 and 0.005 inches per minute or approximately five to six per cent per hour. The test duration was therefore about three hours. Each of the five shale specimens tested at 80 pounds per square inch cell pressure was strained at a different rate. These rates of strain varied from a nominal value of one per cent in 16 minutes to one per cent in 20 hours. The actual rates of strain were very much slower and are discussed in Chapter VI . The three additional shales specimens were strained at a nominal rate of 1.3 x 10^{-5} inches per minute or approximately one per cent every 33 hours. The actual rate of strain before failure was approximately one per cent in 78 hours.

At the completion of a test the angle of the failure plane, if evident, was measured. The specimen was then weighed and immersed in mercury for determination of the volume. Specimens were divided into at least three sections for water content determinations.

5.7 Membrane Tests

When the testing programs described in this thesis were initially under consideration it was recognized that membrane permeability may prove to be one of the severest problems to be faced. Consequently it was decided at the outset to conduct a series of tests to determine experimentally the required arrangement of membranes and seals which would permit the successful conduct of long duration, high pressure, undrained compression tests with pore pressure measurements. The high pressure cell described in section 4.4 and Appendix C was designed and constructed at the University of Alberta specifically to meet this requirement.

The simulated specimen is a three inch long section of 1.5 inch diameter porous Norton tube. This tube was set up as illustrated in FIGURE V.2. A short piece of rubber inner tube was used to cover the pedestal, porous disk and lower end of the Norton stone. The membranes to be tested were then placed over the stone with the aid of a membrane stretcher. They were bound at the bottom with one or more O-Rings and left open at the top extending above the stone by at least one-half inch. The entire system

within the membrane was filled with water draining from a burette through the pedestal. With the water level near the top of the membrane the Lucite cap was placed on the stone, care being taken to avoid trapping air within the system. By this means near saturation was achieved. After sealing the membrane to the top cap with O-Rings, the cell was assembled and filled with deaired water.

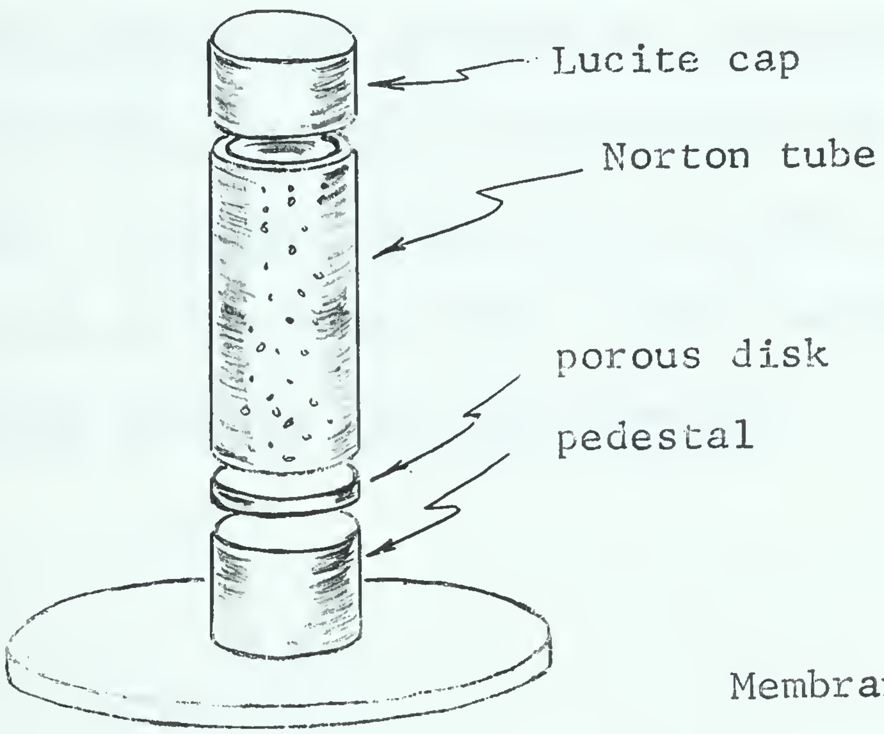

FIGURE V.2

Membrane Test Arrangement

Pressures from 75 to 1500 pounds per square inch were applied to the cell fluid by one of three systems. Originally the pressure was applied directly from a bottle of compressed Nitrogen through a regulator and about eight feet of copper tubing. At a later date a gas/fluid transfer barrier was inserted between the regulator and the cell.

In addition one test was conducted with pressure supplied by the hydraulic apparatus described in section 4.2 Leakage was indicated by a rise of the water level in a 25 ml. burette connected to the cell's drainage port.

The various combinations of membranes and seals tested and corresponding leakage observed are tabulated at TABLE VI.1. In order to verify that the leakage was not occurring between the pedestal and baseplate one test was conducted with the top cap set directly on top of the pedestal. A single rubber dam encircled the contact area and was in turn covered with a rubber membrane and O-Rings. No leakage from the cell was observed.

CHAPTER VI

PRESENTATION AND DISCUSSION OF EXPERIMENTAL RESULTS

6.1 Introduction

This chapter deals with the data and results obtained from the tests described in Chapter V. For clarity of presentation separate sections are devoted to the tests on membranes, Saskatchewan Silt, and Bearpaw Shale although in many respects the three are interrelated. This must be borne in mind by the reader. A short section is devoted to the accuracy which may be expected of the pressure transducer and proposals for its future use.

6.2 Membrane Tests

The successful interpretation of the results of any "undrained" compression test demands that there be absolutely no change in the water content of the specimen during the course of the test. If there is leakage of water either into or out of the specimen the test results may be invalid. This statement is particularly true if pore water pressures are measured.

It would appear that no concerted effort has been made previously at the University of Alberta to determine the extent and influence of membrane leakage even though it has been recognized as a problem elsewhere for many years (Casagrande and Shannon, 1948). The usual procedure has been to encase the specimen with one or two standard prophylactics smeared with silicone grease and sealed with two or more rubber O-Rings. It is probable that if leakage of water into the specimen has not been significant, it is primarily because of the relatively short duration of test required for the soils most frequently tested. It is, however, interesting to note the following comments of Dr. S. Thomson (1963) concerning the testing of Dunvegan clay shales:

> "It would appear that the volume changes as indicated by the burette readings should be the same as that calculated from the initial and final volumes. However, agreement between these two was approximately reached in only 2 of the last 8 consolidated-quick tests. In the remaining tests the discrepancy was 2 to 4.5 c.c. In three of these tests the volume changes showed a decrease by burette readings and an increase by initial and final volume comparison. Just why this anomaly should exist is not known but possible causes could be back pressure application not being equal, a pore pressure existing at dismantling at end of test, clearing the lines of air at various stages of the test and during dismantling ... . There does not seem to be any apparent reason for the discrepancy at this time."

Dahlman (1965) also observed that burette-measured volume changes during triaxial consolidation were about 2 c.c. high when compared with the difference between initial and final volume measurements. He attributed the discrepancy to excess water used to saturate the drainage aids and air trapped between the specimen and membrane. It is the author's opinion, based on the evidence presented herein, that a significant percentage of these discrepancies may be explained by the migration of cell water into the specimen either by leakage through the membrane(s) or past the O-Ring seals or both.

The results of 12 membrane tests conducted as described in section 5.7 are summarized in TABLE VI.1 and illustrated in FIGURE VI.1. The pressure for tests 1 to 6 inclusive was supplied directly by compressed nitrogen through a few feet of water-filled copper tubing. The resulting leakage pattern (FIGURE VI.1) was characterized by an initial burst of a few cubic centimeters of air and water into the burette as the drainage line was opened. This was considered to represent the extraneous water trapped between stone and membrane at the time of set up and was followed by a period of approximately 15 hours during which there was no significant leakage. After

TABLE VI.1 SUMMARY OF MEMBRANE TEST DATA

TEST NO.	MEMBRANE[S]	END SEALS		PRES. PSI.	SOURCE OF PRESSURE	VOLUME EXPELLED C.C. *	PERIOD OF TEST HOURS	AVERAGE RATE C.C./HR.	REMARKS
		TOP	BOTTOM						
1	ONE 1.5" LATEX .007 - .01" THICK	2 O-RINGS	2 O-RINGS	150	NITROGEN	1.90	18.2	0.105	
2	ONE 1.5" LATEX .007 - .01" THICK	2 O-RINGS	3 O-RINGS	150	NITROGEN	5.98	53	0.113	
3	N/A			600	NITROGEN	0			TEST ON PEDESTAL
4	TWO PROPHYLACTICS W/ HIGH VAC. GREASE	2 O-RINGS	2 O-RINGS	150	NITROGEN	2.90 22.70	5.9 18.3	0.49 1.24	SMALL HOLE IN OUTER MEMBRANE
5	TWO PROPHYLACTICS W/ HIGH VAC. GREASE	2 O-RINGS	2 O-RINGS	75	NITROGEN	1.40	29.8	0.047	
6	ONE 1.5" LATEX	**	**	200	NITROGEN	4.60	41.7	0.11	FILTER PAPER ON STONE
7	ONE 1.5" LATEX	**	**	150 TO 1000	HYDRAULIC	0.04	239	1.68×10^{-4}	FILTER PAPER ON STONE
8	ONE 1.5" LATEX	2 O-RINGS	2 O-RINGS	1000	NI. WITH BARRIER	1.81	166	1.09×10^{-2}	
9	TWO 1.5" LATEX ONE LAYER OF SARAN	3 O-RINGS	3 O-RINGS	1000	NI. WITH BARRIER	6.00	19.5	0.308	
10	TWO 1.5" LATEX ONE LAYER OF SARAN	ELASTIC BANDS	ELASTIC BANDS	1000 TO 1400	NI. WITH BARRIER	0.68 0.39	41.8 29.0	1.63×10^{-2} 1.35×10^{-2}	SILICONE OIL SOAKED
11	TWO 1.5" LATEX	ELASTIC BANDS	ELASTIC BANDS	1000	NI. WITH BARRIER	0.90	25.5	3.53×10^{-2}	SILICONE OIL SOAKED
12	TWO 1.5" LATEX WITH SARAN	ELASTIC BANDS	ELASTIC BANDS	1000	NI. WITH BARRIER	-0.45	118.8	-3.79×10^{-3}	SILICONE OIL SOAKED

* AFTER INITIAL 6 SECONDS

** NOT RECORDED

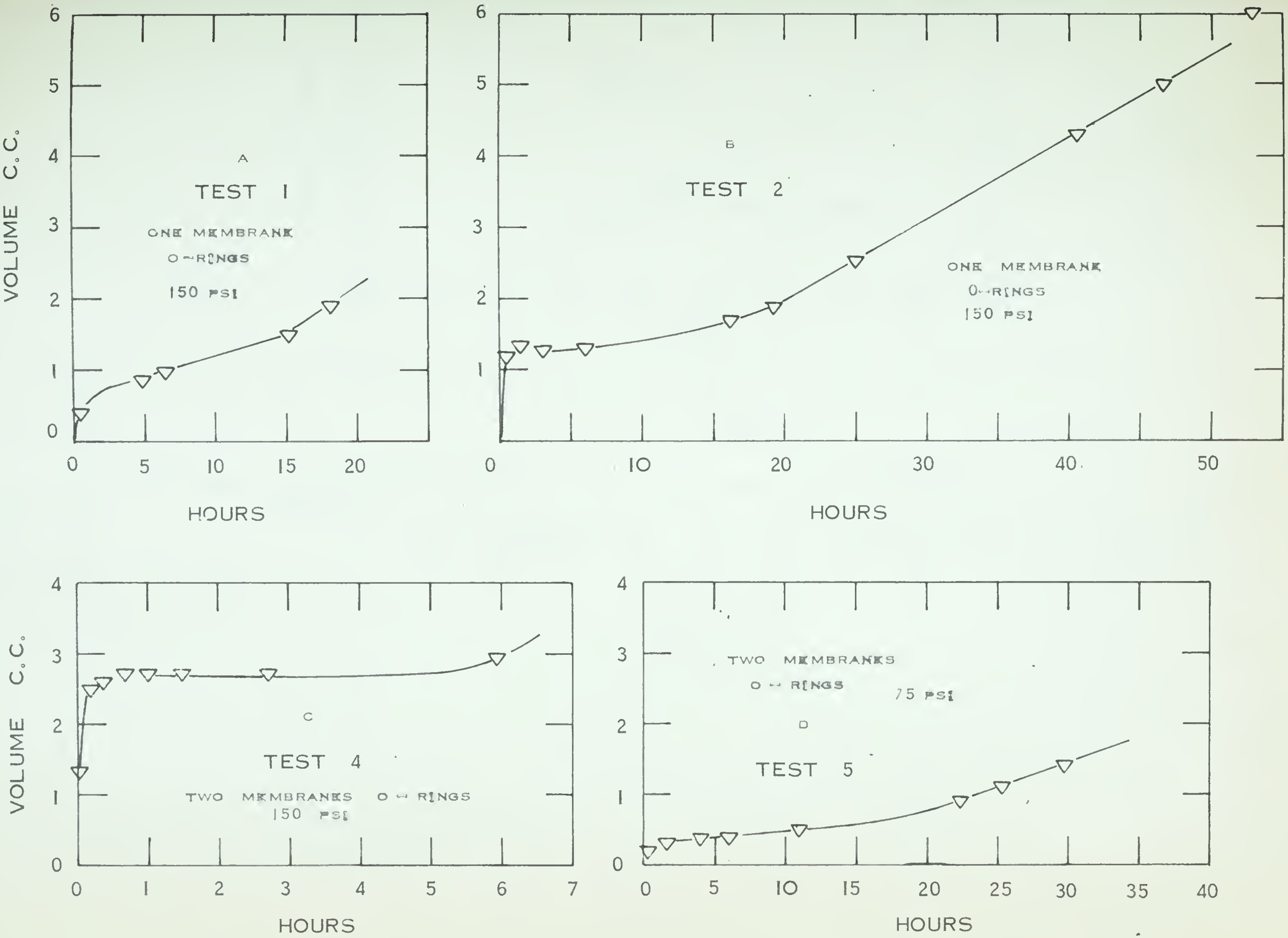

FIGURE VI.1 VOLUME OF LEAKAGE - MEMBRANE TESTS

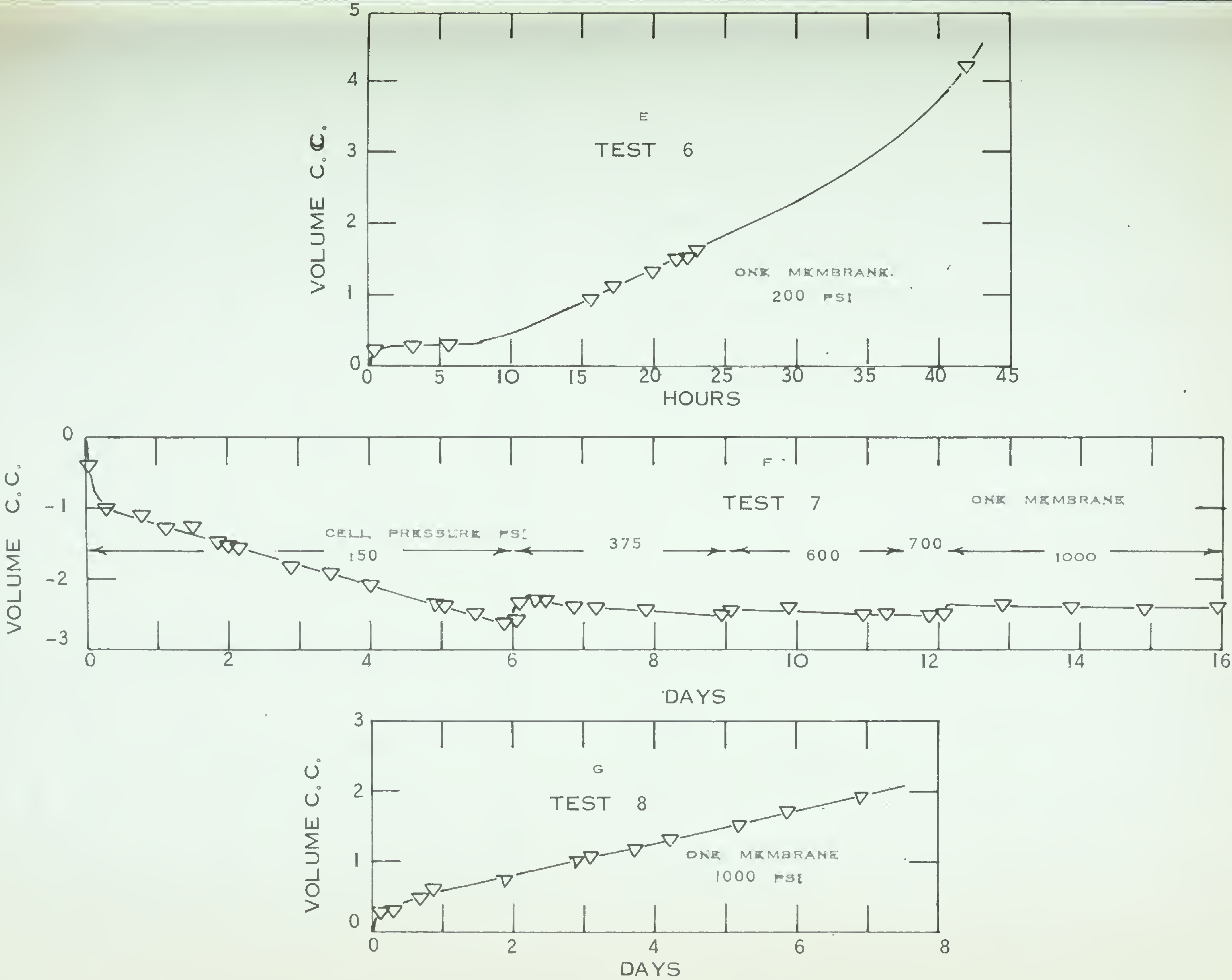

FIGURE VI.1 VOLUME OF LEAKAGE - MEMBRANE TESTS

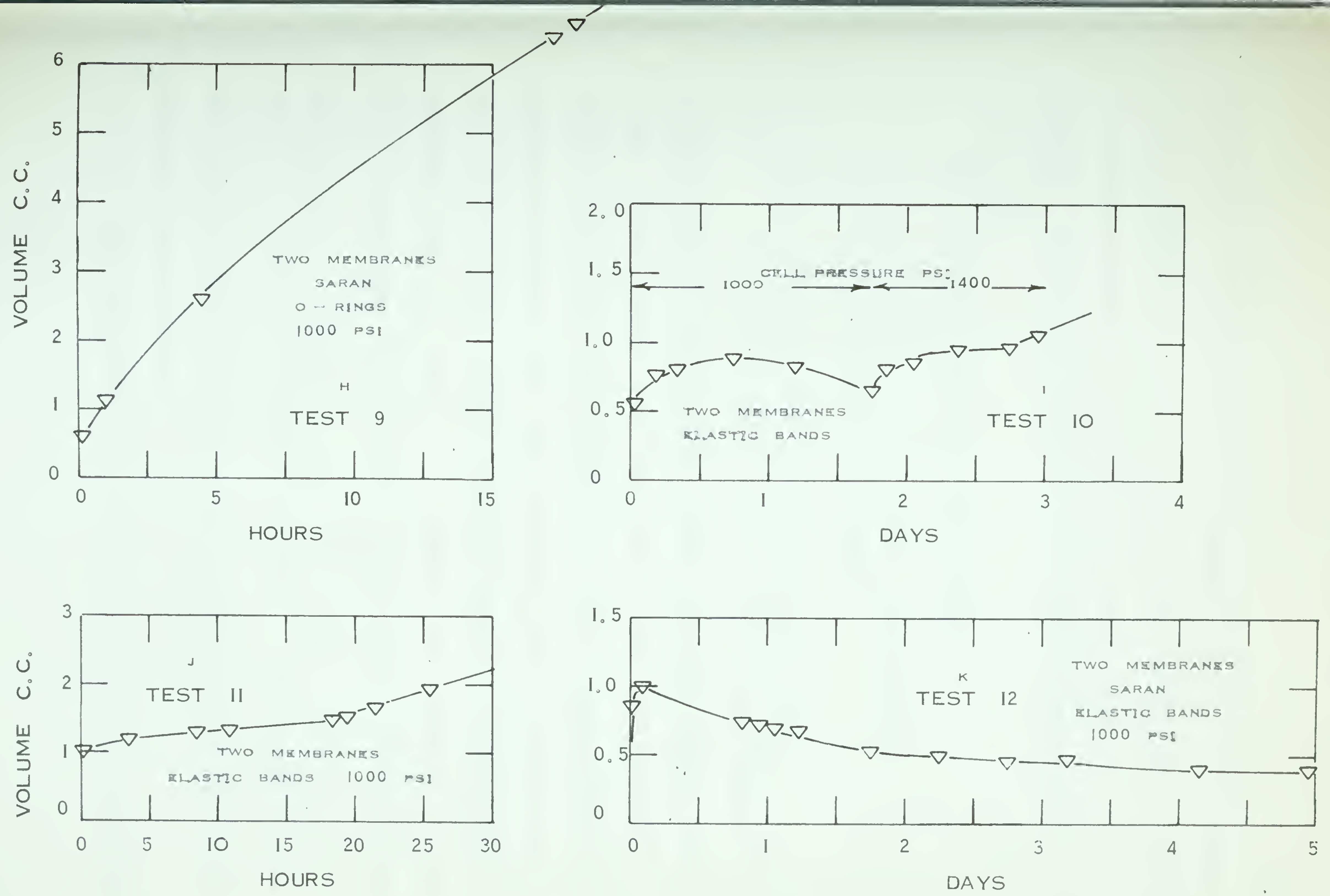

FIGURE VI.1 VOLUME OF LEAKAGE - MEMBRANE TESTS

the 15 hour "break-out" point leakage occurred at a steady rate of about 0.1 to 0.2 c.c./hr. Upon examination none of the combinations tested showed any obvious reason for the leakage. The possible sources of trouble were thought to be as follows:

(a) leakage past the 0-Ring seals,

(b) diffusion of gas through the membrane,

(c) minute holes in the rubber,

(d) leakage between the pedestal and base.

In the opinion of Olson (1964b) the 0-Rings do not normally cause trouble; the leakage from this source being "many hundred times less than that through the membrane". There is, however, the possibility that if the membranes are larger in diameter than the specimen, the cell fluid may pass into the specimen through channels caused by pinching the membrane with the 0-Ring. In view of the thickness of some of the membranes tested (0.007 - 0.01 inches) and the apparent good quality, leakage through minute holes formed in the rubber during molding was considered unlikely. The possibility of cell water passing the 0-Ring seal between the pedestal and baseplate was subsequently disproved by test 3.

The "permeability" of a medium to air may be defined

as follows (Poulos, 1964):

$$K_s = \frac{sD}{p_s}$$

where s = solubility of air in the medium ($cm.^3$ of air per $cm.^3$ of medium),

D = coefficient of diffusion ($cm.^2$/sec.),

p_s = standard atmospheric pressure (gm./$cm.^2$).

Since the solubility of air in water is 0.019 at 22°C. and the coefficient of diffusion of nitrogen or oxygen in water is about 2×10^{-5} $cm.^2$/sec. at 22°C. (Washburn, 1926), the permeability of water to air is

$$K_s = \frac{sD}{p_s} = \frac{0.019 \times 2 \times 10^{-5}}{1033} = 3.7 \times 10^{-10} \text{ cm.}^4\text{/gm.sec.}$$

Further, since the unit weight of water is approximately one gm./$cm.^3$ the coefficient of permeability of water to air may also be expressed as

$$k = 3.7 \times 10^{-10} \text{ cm./sec.}$$

According to Poulos the permeability of natural rubber to air is 4.6 times less than the permeability of water to air.

Assuming the worst possible condition (i.e. no membrane and cell water saturated with nitrogen) it may be

seen that the maximum flow which could occur through three feet of water-filled copper tubing under a partial pressure differential of 200 p.s.i. would be less than one cubic millimeter per day. In contrast, Olson (1964c) reports direct nitrogen leakage through two membranes of up to 720 millilitres per day at 100 p.s.i. and for nitrogen in solution in water, a leakage rate of the order of 50 millilitres per day. Since there was at least three feet of water between the cell and nitrogen bottle and since the rates of leakage were not of the same order as those observed by Olson, it would appear highly unlikely that the leakage noted in tests 1 to 6 inclusive can be attributed directly to the use of nitrogen. In any event pressure for the remaining tests was supplied either hydraulically or through a transfer barrier so that diffusion of gas was eliminated as a possible source of trouble.

While the rate of leakage was significantly reduced when the transfer barrier was used, it was still undesirable. Furthermore, those systems which appeared to offer promise when tested in the membrane cell continued to give trouble when applied to shale specimens in the high pressure apparatus.

Tests 7 and 12 indicated a net decrease in volume of water expelled. With regard to test 7 this anomaly may

be partly explained by the fact that the cell water was initially at a temperature of 80 - 90 degrees centigrade and subsequently cooled to room temperature. The remainder of the decrease in both tests 7 and 12 exceeded that which could have been expected from evaporation or temperature fluctuations. No reasonable explanation for the apparent anomaly of water flowing against a pressure gradient can be offered at this time.

The data of TABLE VI.1 do not appear to offer any correlation between rate of leakage and number or treatment of membranes and bindings used or pressure differentials applied. In fact, there are insufficient data on which to base any firm conclusions regarding the major source of leakage although in the author's opinion the bindings must be suspected.

As will be appreciated, all of the shale specimens which failed to consolidate under the applied cell pressure offered further evidence of membrane leakage. Shale specimen number eight was particularly notable in this regard. It was sealed with two English membranes, a wrap of Saran, and an layer of Silicone grease. A cell pressure of 600 p.s.i. was applied and slightly over 25 per cent of the water in the specimen was expelled over a period of three days. There was no indication that the rate of drainage (two cubic centi-

meters over the last 24 hours) was slowing down at the end of this time. The drainage valve was therefore closed and the build-up of pressure within the membranes was monitored with the transducer. The increase of internal pressure with time is shown in FIGURE VI.3.

The essentially constant pressure during the initial three hours probably reflects the presence of air within the system and does not therefore preclude the possibility that leakage was occurring. Once saturated the internal pressure increased to 290 p.s.i. in about six hours and then, at a slower rate, to a value of 425 p.s.i. before the test was discontinued. The distribution of moisture contents in the specimen at con clusion of the test (FIGURE VI.2) tends to substantiate that the O-Ring seals are the most likely source of leakage.

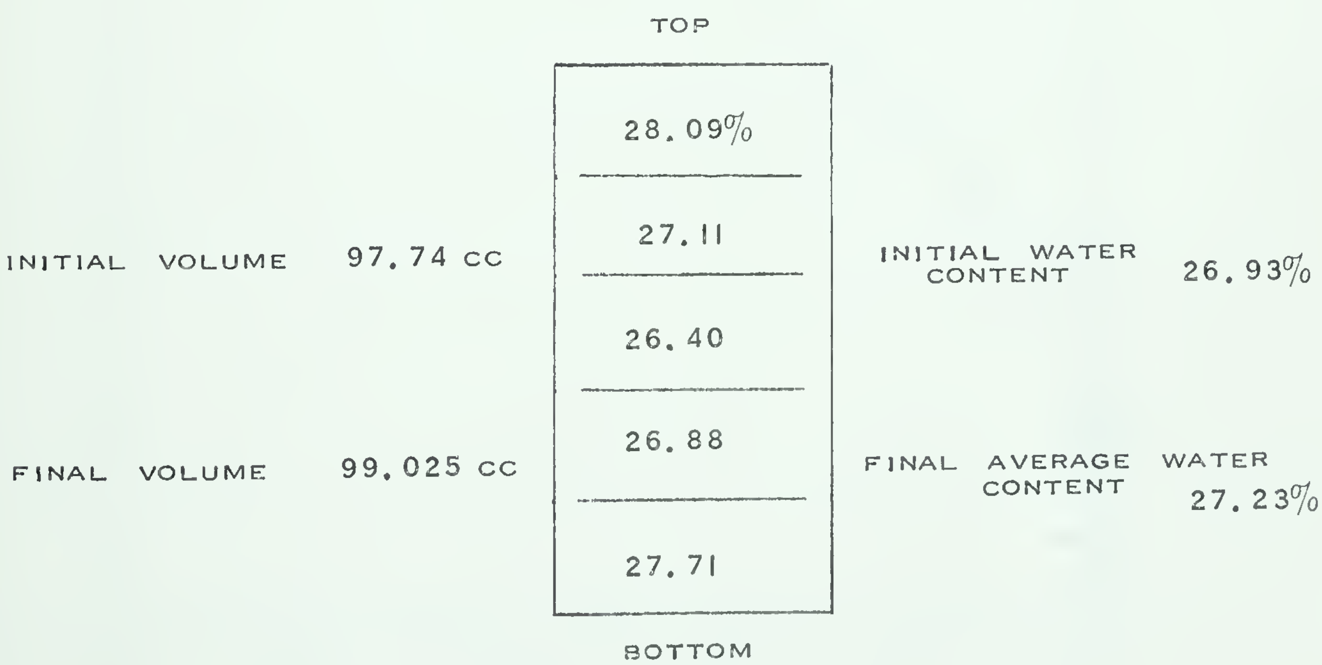

FIGURE VI.2 Water Content Variation at End of Test
Shale Specimen No. 8

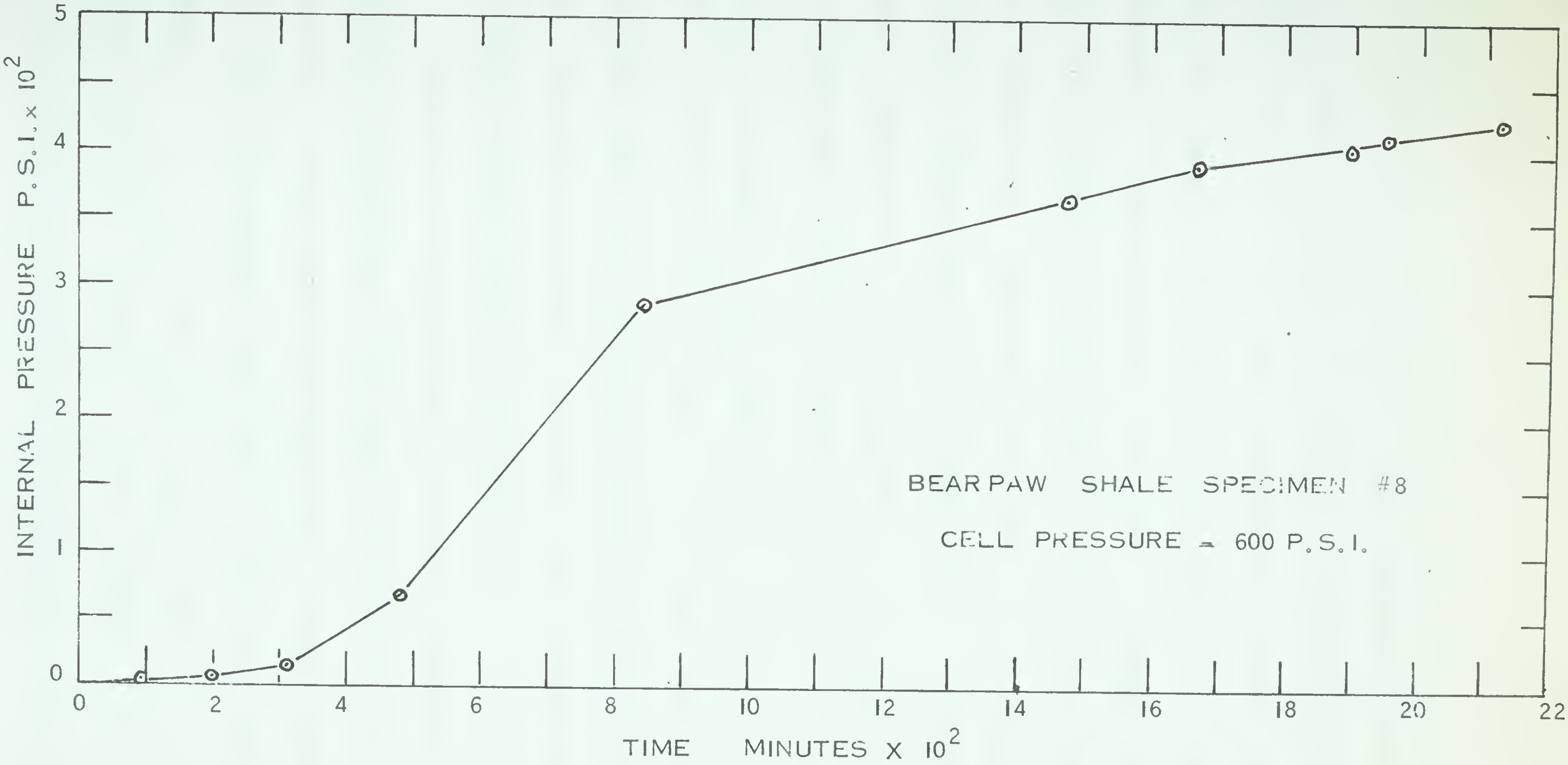

FIGURE VI.3 BUILDUP OF INTERNAL PRESSURE DUE TO MEMBRANE LEAKAGE

It may be argued that three days is an insufficient period to expect 100 per cent primary consolidation to be approached in such an impermeable specimen. However, another specimen was later set up under a cell pressure of 1000 p.s.i. and drainage was permitted for 16 days. Although the rate of drainage was continually decreasing, there was no indication that the "time curve" was about to level out at the end of this period. Furthermore, the volume expelled represented over 50 per cent of the initial moisture contained in the specimen. In addition, there is fairly good evidence (section 6.5) that values of t_{100} for the shale at 80 - 120 p.s.i. are in the order of 200 to 1000 minutes. Since the coefficient of consolidation does not generally decrease a great deal with pressure there is no reason to believe that values of t_{100} should exceed 20,000 minutes at 1000 p.s.i.

Despite the foregoing, two shale specimens were subsequently consolidated and tested successfully at 120 p.s.i. cell pressure. The essential difference in sealing these specimens was that two O-Rings were placed below the rubber dam on the pedestal. It would therefore appear that the rubber dams may in fact encourage leakage past the bindings instead of reducing it as intended. Greasing the pedestal and top cap has

been recommended by a number of investigators (Crawford, 1963; Poulos, 1964) as a means of reducing binding leakage. This technique was not employed during the course of this research and is certainly one aspect which should be investigated further.

As a consequence of the experiences with membrane leakage reported in the above paragraphs, a new concept in rubber jackets for shale specimens has developed (Brooker, 1965). This jacket, which is illustrated in FIGURE VI.4, virtually eliminates the possibility of binding leakage and should considerably reduce, if not completely eliminate, the flow of water through the rubber under high cell pressures. It appears to be very promising on the basis of tests to date. However, techniques for using it on actual specimens must be developed; particularly with regard to saturating the system. Also, its influence on the strength of shale specimens must be evaluated.

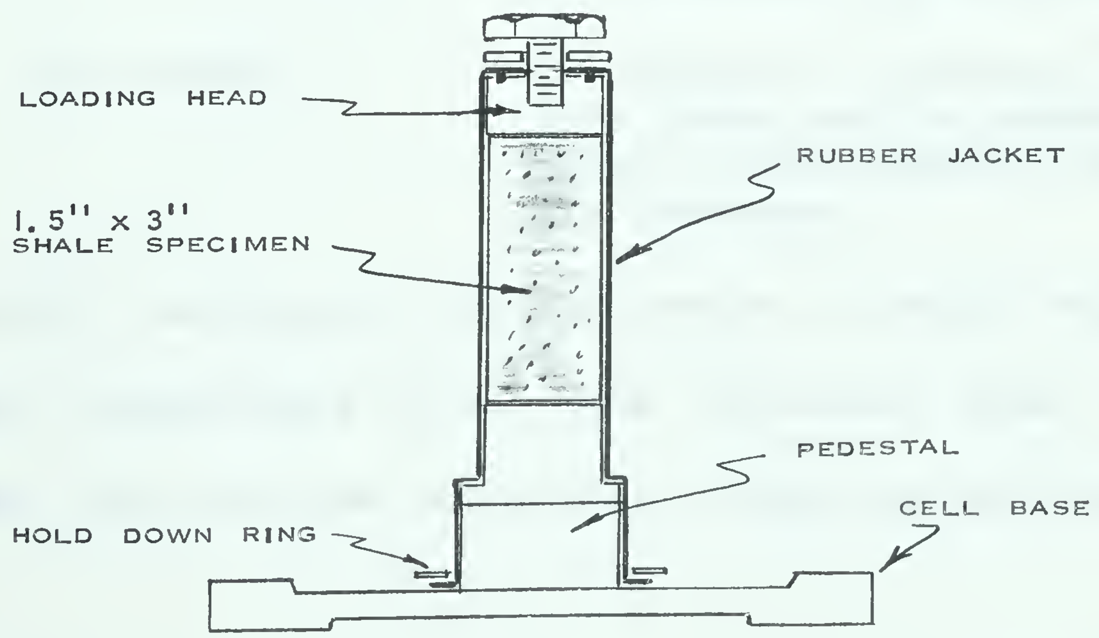

FIGURE VI.4 Rubber Jacket for Shale Specimens

6.3 The Pressure Transducer

The features and calibration of the transducers as well as the techniques developed for employing them have been described briefly in Chapter IV and more fully in Appendix B. It remains, therefore, only to discuss the accuracy of pore pressure measurements which may be expected from their use.

To permit an understanding of the performance reliability of the transducer reference is made to FIGURE VI.5. The total output error is considered to be the sum of the non-linearity and hysteresis at any point. These terms are defined as follows:

(a) Non-linearity: the deviation between the calibration curve and a straight line joining the points of zero and full scale pressure output

(b) Calibration Curve: that curve passing through the average measured values of output at any pressure as the pressure is increased and decreased

(c) Hysteresis: the deviation between the calibration curve and two separate data points corresponding to a specific pressure.

Repeatability, defined as the difference in output between successive calibrations of the same instrument under identical conditions, must also be considered. When the calibration

data for the APT-25 transducer, recorded with a Baldwin type L strain indicator, are plotted in the form of FIGURE VI.5 the sum total of these deviations is so small that it cannot be easily detected. The strain indicator, however, provides extreme sensitivity. Full scale output corresponds to about 10,000 microinches per inch. Therefore non-linearity and, in the case of the 300 p.s.i.a. transducer, hysteresis are apparent when the calibration data are plotted in the form shown in Appendix B. Although this form of calibration curve necessitates additional calculations, it is the best means of taking advantage of the transducer's inherent accuracy.

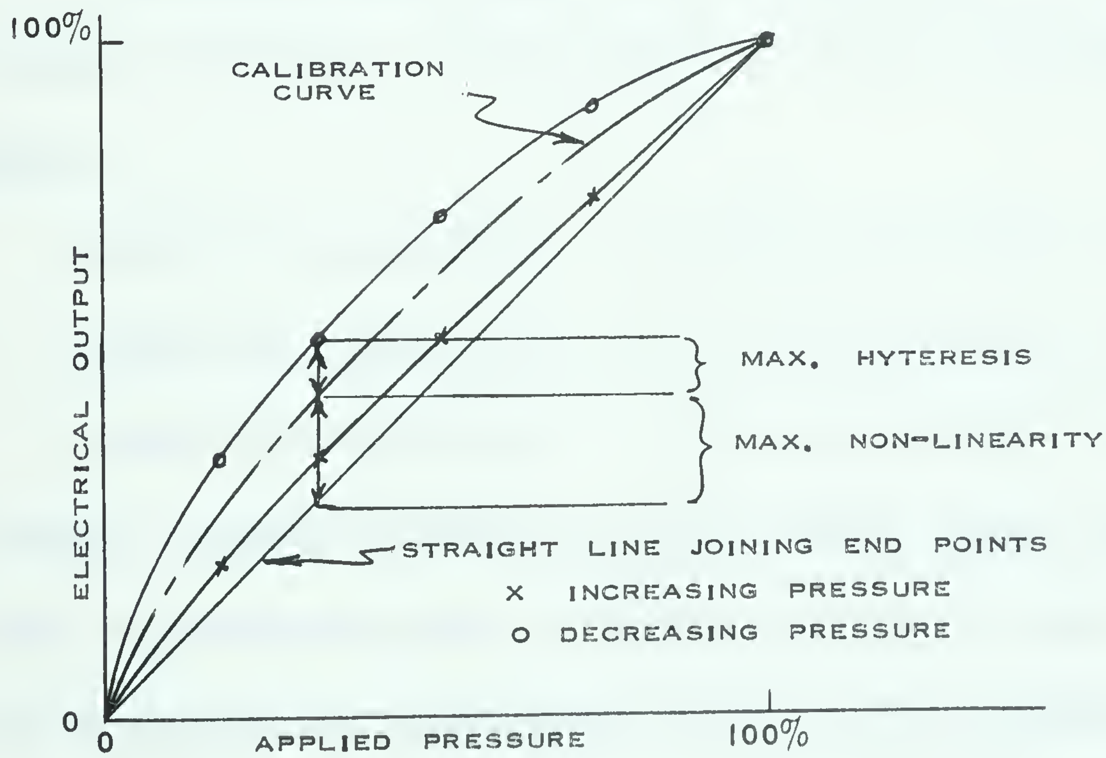

FIGURE VI.5 Illustration of Calibration Definitions

The paradox of the transducer-strain indicator recording system is that the precison of readings is far greater than the accuracy of the pressure measured or of any other measurement made during the conduct of a test. Thus it may lead to a false sense of accuracy unless the limitations of the recording system are constantly borne in mind. For example, over the range from 0 to 100 p.s.i. the pressure on the diaphragm can be readily estimated to plus or minus 0.02 per cent of full scale. This precision is unnecessary when the influences of partial saturation, non-equalization, and even the plotting of pore pressure data are considered.

The accuracies which can be expected of the transducer-strain indicator systems used in this investigation are as follows:

APT25-1C transducer: 0.15 per cent full scale

APT25-3C transducer: 0.5 per cent full scale

APT25-2M transducer: 0.18 per cent full scale.

These values assume no error in the Bourdon gauges against which the transducers were calibrated and full power supply. They are primarily an expression of the non-repeatability of the recording system and should be confirmed by periodic recalibration with a dead weight gauge tester. The APT25-1C

instrument was, in fact, recalibrated by means of a dead weight tester at a later date. The maximum difference between the two calibration curves was less than 0.35% full scale.

It appeared that the most desirable method of assessing the transducer's reliability and thus gaining confidence in its use would be by direct comparison with a proven pore pressure system under test conditions. The results of this comparison are illustrated in FIGURE VI.6. It is considered that the agreement shown is as good as may be anticipated.

The volume factor of the PT25 transducer under conditions similar to those used in this investigation is 0.4×10^{-6} in.^{3}/p.s.i. (Richardson and Whitman, 1963). By comparison the range in volume factors reported for the null indicator complete with 20 feet of copper tubing is 5.0 to 7.4×10^{-6} in.^{3}/p.s.i. (Bishop and Henkel, 1962). Thus the maximum volumes which are required to register 80 p.s.i. pore pressure are 5.2×10^{-4} c.c. in the case of the transducer and 0.01c.c. in the case of the null indicator. This difference would not account for any appreciable percentage of the disagreement in shearing resistance noted in FIGURE VI.6.

Apart from the superior accuracy and low volume factor of the transducer its prime advantage lies in the fact that it

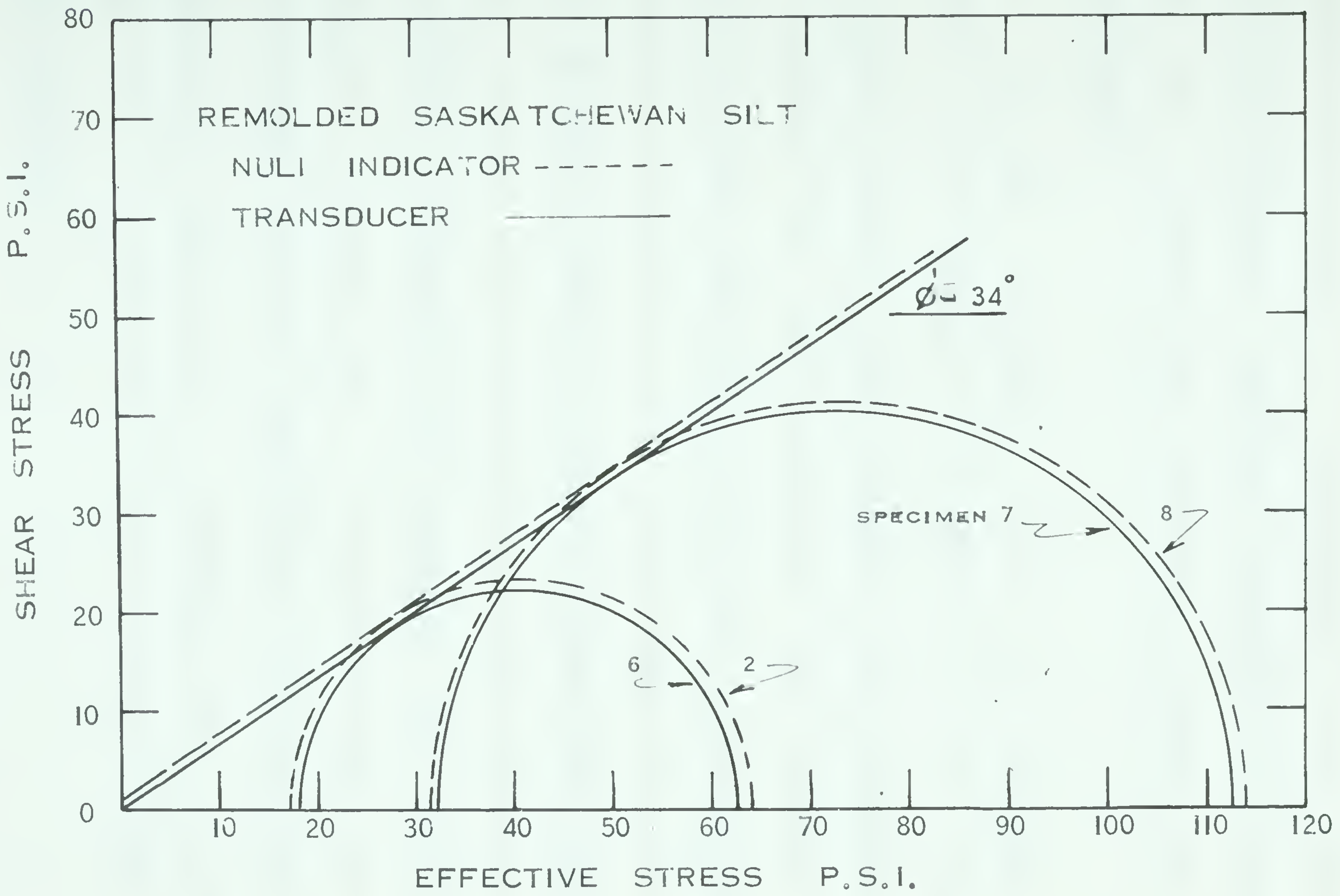

FIGURE VI.6 COMPARISON OF PORE PRESSURE MEASURING SYSTEMS

need not be continually adjusted by a technician. Other pore pressure measuring systems (Penman, 1953; Raymond, 1963) enjoy this capability but only at the expense of some other feature such as slow response. Raymond's "modified thermometer", for example, has a volume factor 2000 times greater than that of the transducer.

In order to properly utilize the advantage of self-adjustment the recording system should be fully automated. As an interim measure a semi-automatic data acquisition system may be created which would provide for the continuous monitoring of several load cells, LVDT's and pressure transducers. The external circuitry of this system might include a guarded take-off DC power module to ensure constant excitation voltage, a digital readout micro-voltmeter or strain indicator, a heavy resistor to permit direct reading, and a multichannel switching and balancing unit. To fully automate the system this arrangement would be replaced by a multichannel digital scanning system which would continuously scan and print out the data from up to 200 instruments in terms of load, strain and pore pressure.

6.4 Saskatchewan Silt

(a) Assessment of Errors

Twelve silt specimens were trimmed and mounted for testing. The data of three tests were subsequently discarded for reasons of either damage to the specimen or difficulties with the equipment. Results of the remaining nine tests are tabulated at TABLE VI.2.

The very soft consistency of the specimens made it difficult to accurately determine their dimensions without causing damage. For this reason the void ratios reported in TABLE VI.2 are not more accurate than $\pm$ 1.5 per cent. This figure represents the estimated cumulative effect of errors in all weights and measurements associated with the determination of void ratio. The loads, strains and pore pressures measured during the conduct of each test are considered to be accurate within $\pm$ 1.0 per cent. Since loads were determined from the actual proving ring dial readings and not from the differences in readings, the influences of variation in proving ring calibration and piston friction are reduced. Piston friction associated with the high pressure cell was checked by comparing the external load recorded by a proving ring with the internal force provided by the cell pressure. No

TABLE VI.2 SUMMARY OF TEST DATA – SASKATCHEWAN SILT

SPEC.	ORIG. M.C. %	INITIAL VOID RATIO	INITIAL s %	CELL PRES. PSI.	BACK PRES. PSI.	B	ε_F %	$[\sigma_1-\sigma_3]_F$ PSI.	$\bar{A}_F$	e_F	FINAL s %	SHEAR PLANE ANGLE	PORE PRES. MEAS'D BY
2	22.35	.65	92.0	30	45	.79	11.8	47.09	.276	–	–	SHEAR AND BULGE	NULL INDICATOR
6	24.10	.655	98.3	30	40	.90	20	44.75	.269	.577	100	BULGE	APT25 – 1C
7	24.22	.648	99.8	60	40	.915	20	81.08	.35	.547	101.3	BULGE	APT25 – 1C
8	23.93	.634	100.8	60	40	.92	20	83.06	.348	.544	100.8	BULGE	NULL INDICATOR
11[2]	23.27	–	96.9	225	30	.375	12.2	222.0	.615	.536	99.1	SPLIT AND BULGE	APT25 – 3C
12	24.82	.666	99.5	350	30	.30	11.0	308.2	.751	.521	101.5	58	APT25 – 3C
13	25.13	.670	99.8	750	30	.24	11.0	584.3	.895	.485	103.9	63	APT25 – 2M
14	24.95	.664	100.3	1203	50	.47	12.0	886.5	.98	.449	101.8	59	APT25 – 2M
15	25.28	.633	106.7*	1500	50	.912	10.0	1069.3	1.04	.436	100.6	55	APT25 – 2M

* INDICATIVE OF ERROR IN MEASUREMENT OF SPECIMEN DIMENSIONS

eccentricity of load was applied. The friction furnished by the O-Ring seal around the piston reduced the external load by less than one per cent at pressures up to 1400 p.s.i. .

(b) Triaxial Consolidation Characteristics

The silt specimens consolidated very rapidly (FIGURE VI.7). Values of t_{100} for the specimens tested at 30 p.s.i. and 60 p.s.i. varied from 30 minutes to 110 minutes with no consistant trend. Those consolidated at the higher cell pressures (225 to 1500 p.s.i.) achieved full consolidation under each increment within two to eight minutes. Again no trend between t_{100} and consolidating pressure is apparent. The only reasons that can be offered for the remarkable reduction in consolidation time are that the "high pressure" specimens (number 11(2) to 15 inclusive) were molded from a different batch of dry soil and to a slightly higher effective pressure. These differences are not borne out, however, by the initial void ratios (TABLE VI.2) which are all within $\pm$ 2.6 per cent of 0.65.

Advantage was taken of the rapid consolidation times by applying the higher cell pressures in increments. This provided additional data for the void ratio-log pressure

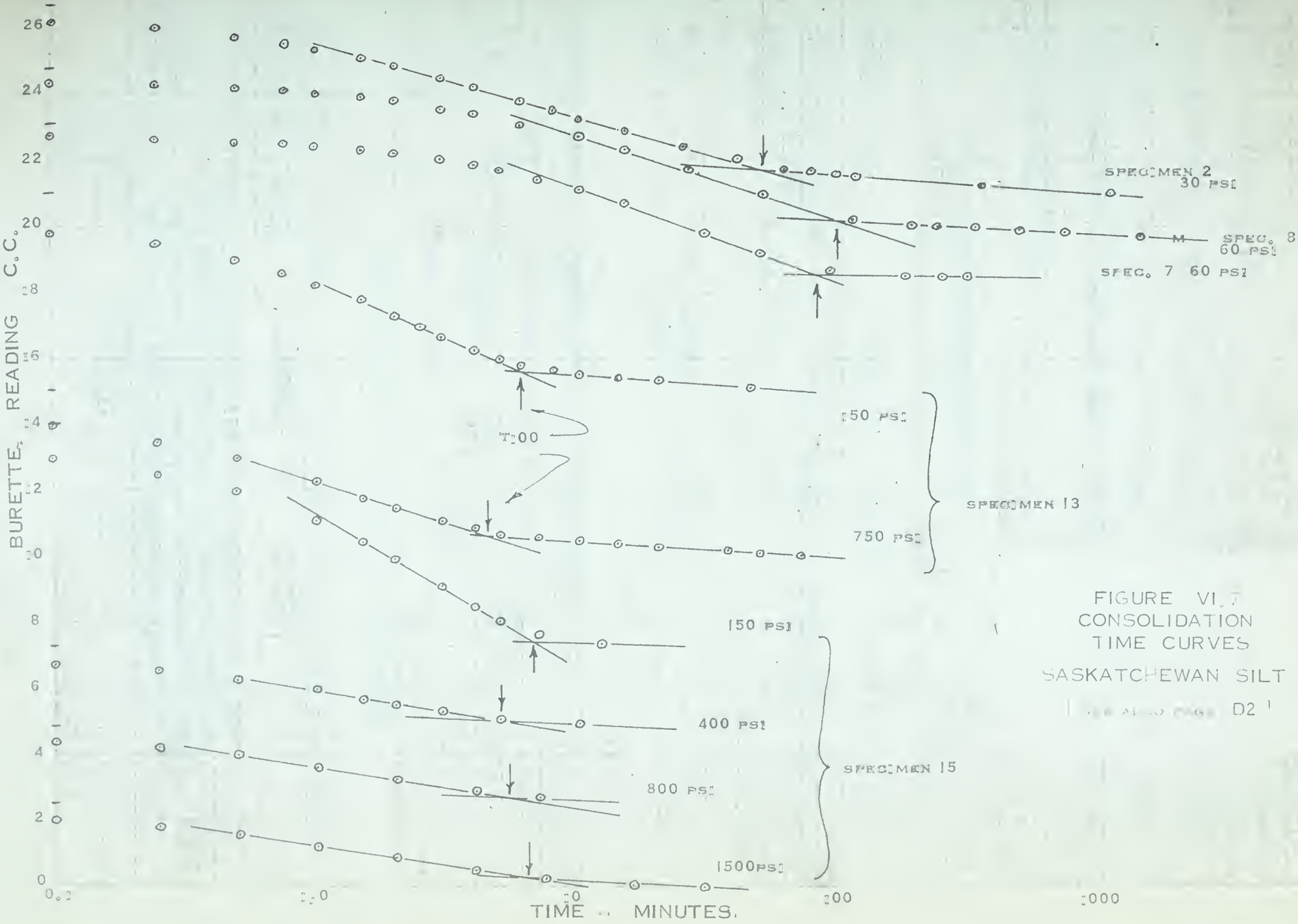

FIGURE VI.7
CONSOLIDATION
TIME CURVES
SASKATCHEWAN SILT
(See also page D2)

curve (FIGURE VI.8). While the data appear to fit a straight line as would be expected for normally consolidated soil, the scatter is somewhat greater than might be anticipated. The deviations are of the same order as the estimated possible error in void ratio.

(c) Pore Pressure Reaction

According to Skempton (1954) the reaction of pore pressure to an instantaneous increase in cell pressure should be 100 per cent if the specimen is saturated. This reaction is expressed as:

$$B = \frac{\Delta u}{\Delta \sigma_3} = \frac{1}{1 + \eta \frac{C_v}{C_c}}$$

where Δu = increment of pore pressure

$\Delta \sigma_3$ = increment of cell pressure

η = porosity

C_v = compressibility of the fluid

C_c = compressibility of the soil structure

B = 1 when the degree of saturation is 100 per cent because in Skempton's words:

> "In saturated soils (zero air voids) C_v/C_c is approximately equal to zero, since the compressibility of water is negligible compared with that of the soil structure."

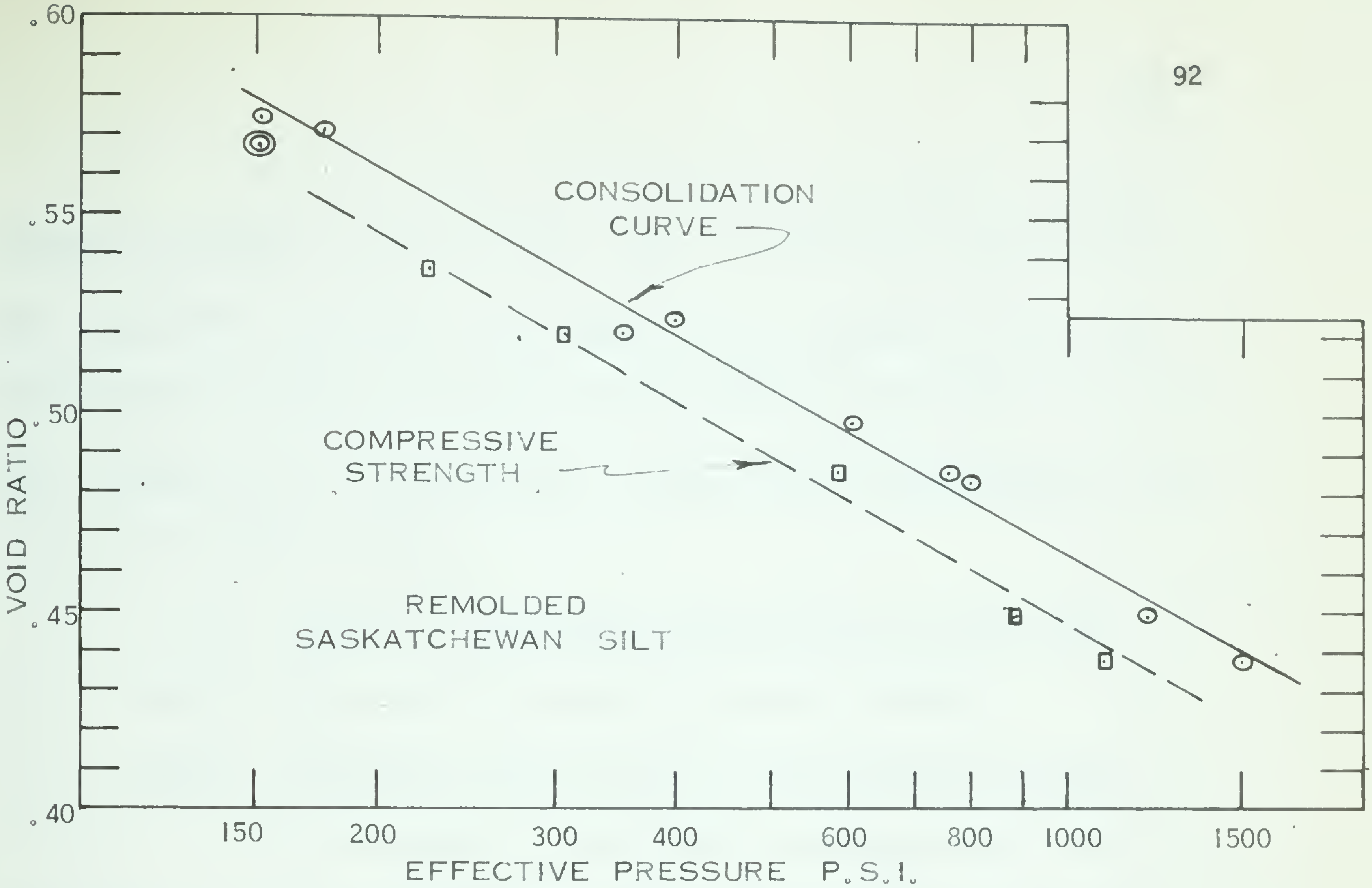

FIGURE VI.8 SUMMARY OF TEST RESULTS

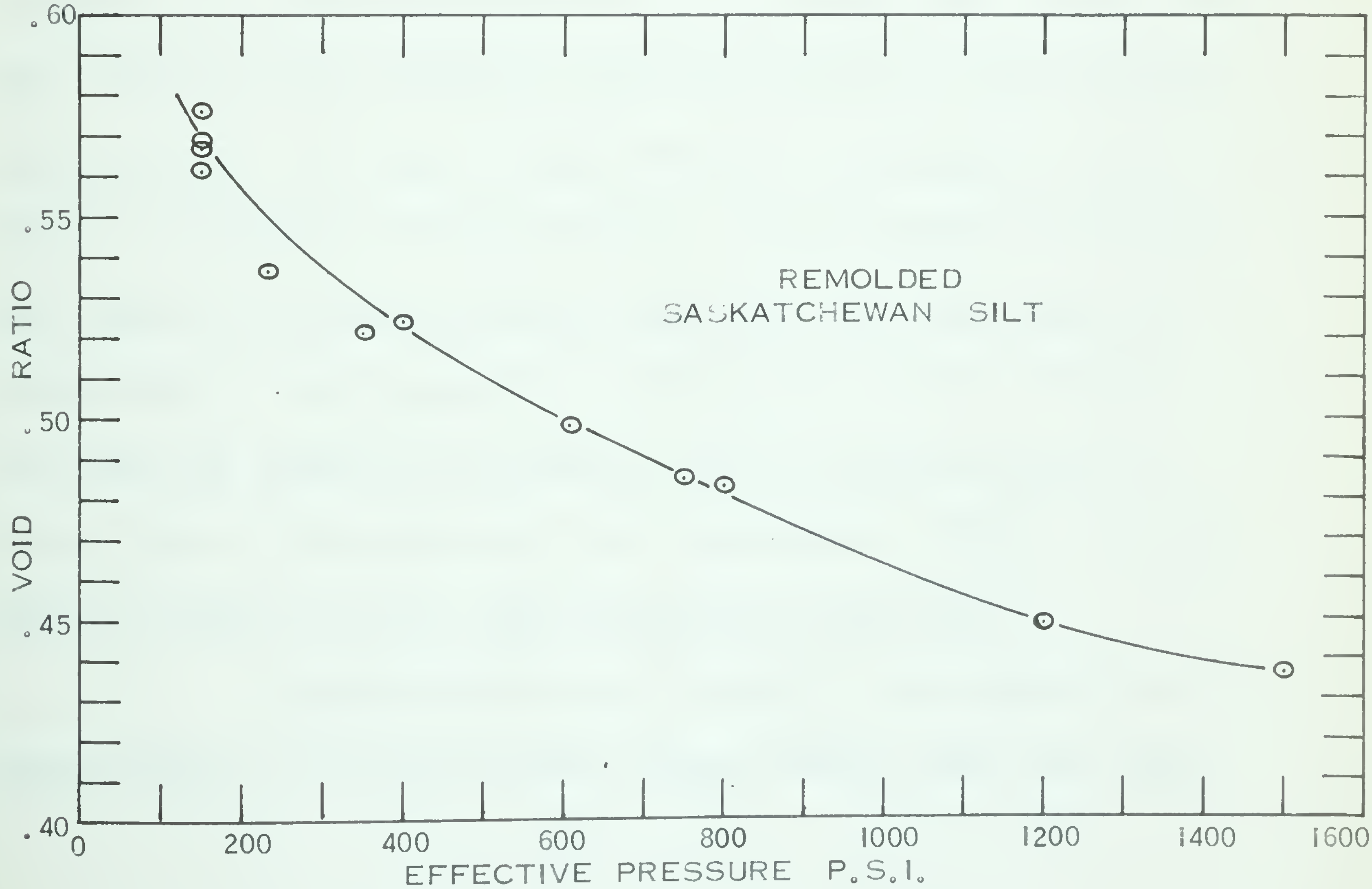

FIGURE VI.9 PRESSURE – VOID RATIO CURVE

Of the nine tests recorded in TABLE VI.2 the highest reaction exhibited was 0.92. Reactions exhibited by the specimens tested at the higher cell pressures (with the exception of number 15 which may have been influenced by binding leakage) were notably poor. This may be attributed to one or more of three causes:

(i) degree of saturation less than 100 per cent

(ii) free air in the pore pressure system

(iii) assumption that C_v/C_c is approximately zero.

With regard to the first possible cause, it is considered that the percentages of full saturation reflected in TABLE VI.2 (with the exception of specimen 15) represent the true state of the specimens within the bounds of the accuracy noted earlier. In view of the method of preparation and subsequent treatment of the specimens, it is difficult to imagine the degree of saturation being much less than that reported. Furthermore, a back pressure of about 25 p.s.i. is adequate to raise the degree of saturation from an initial value of 96 per cent to 100 per cent (Lowe and Johnson, 1960). In each case at least 30 p.s.i. back pressure was applied.

The possibility of free air in the pedestal and porous plate is an ever-present danger despite the care taken

to avoid this situation. It is considered that most free air would be expelled during the initial stages of consolidation under several hundred pounds per square inch pressure or driven into solution by the back pressure. The use of embedded ceramic disks which are now available for the high pressure cells will help to eliminate this possibility.

The assumption that the ratio of soil structure compressibility to that of water is approximately zero holds true for most soils at normal pressures. For certain soils (see section 6.5(c)) and at high confining pressures for others this assumption may not be correct. If the data of FIGURE VI.8 are replotted with pressure on a natural scale (FIGURE VI.9) the slope of the curve at any point may be taken to be the coefficient of compressibility, $a_v = \frac{\Delta e}{\Delta p}$. From this ratio the change in volume per unit volume per unit pressure change, $m_v = a_v/1 + e$, may be determined for any point on the curve. Thus at 75 p.s.i. the drained compressibility of the soil structure is 372×10^{-6}/p.s.i. . This is about 100 times greater than the compressibility of water and the assumption holds. Within the range 1200 to 1500 p.s.i., however, the compressibility of the structure is reduced to 30×10^{-6}/p.s.i., or less than 10 times that of water. The theoretical maximum

reaction for the silt at these pressures would be about 96.5 per cent. At pressures above 1500 p.s.i. it is probable that the compressibility of the structure approaches that of water. The maximum reaction would then depend entirely on the porosity.

The same general conclusion may be reached by comparing modulii of elasticity. The bulk modulus of water is approximately 300,000 p.s.i. (Addison, 1956). By comparison, the initial tangent modulus of the Saskatchewan Silt (FIGURE VI.10) would appear to be 100,000 p.s.i. . If a value of Poisson's ratio equal to 0.35 is assumed the bulk modulus of the silt is approximately 110,000 p.s.i. . Thus the ratio of compressibilities is reduced to 2.7 and the maximum value of B, to about 0.90. This is obviously not a complete explanation for the poor reaction observed but is considered a significant factor.

(d) Stress-Strain Characteristics

The data, computations, and stress-strain curves for a typical test are shown at Appendix D. A composite plot of the deviator stress-strain curves for cell pressures from 225 p.s.i. to 1500 p.s.i. is shown at FIGURE VI.10. It will

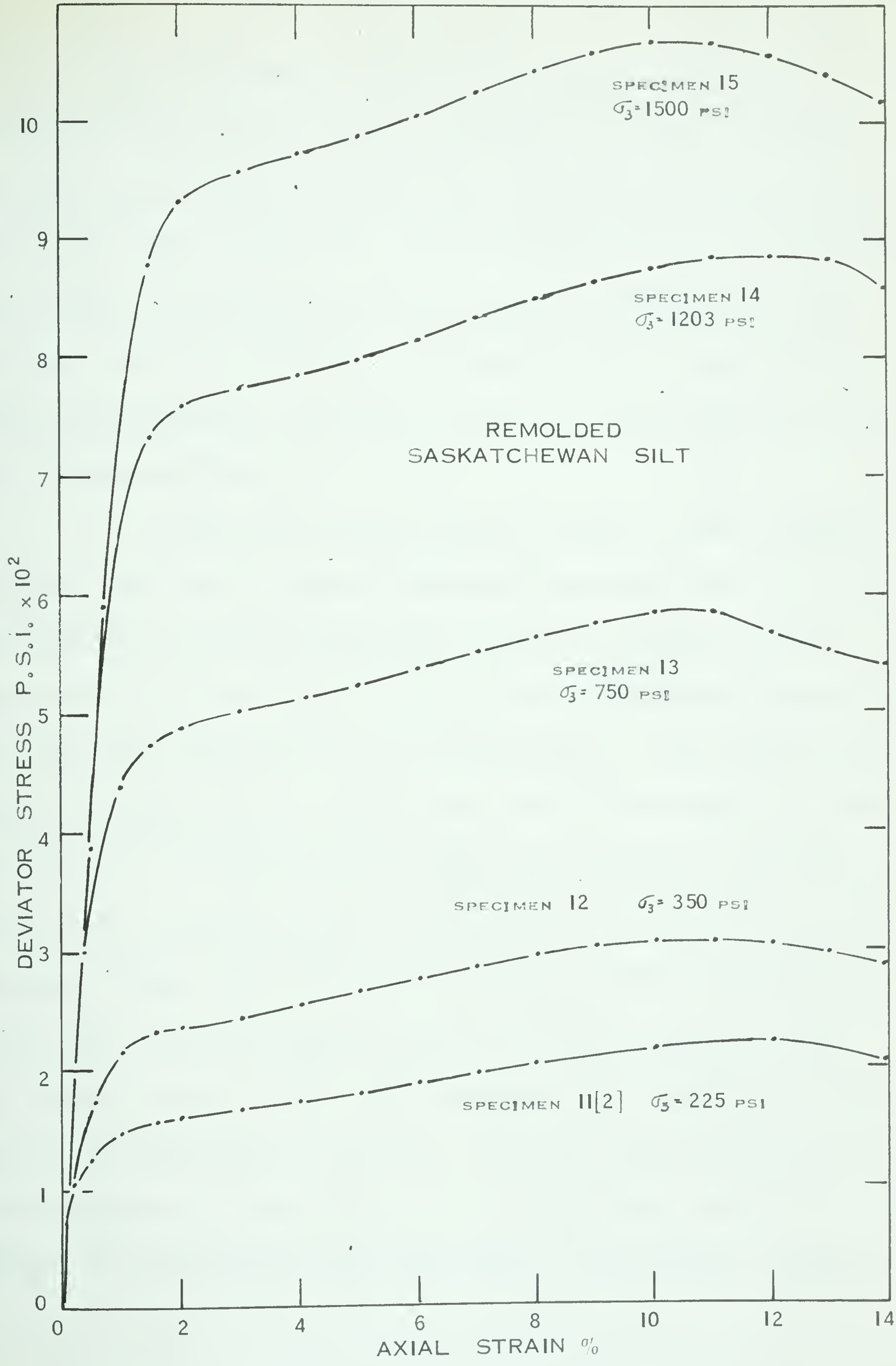

FIGURE VI.10 STRESS -- STRAIN CHARACTERISTICS
HIGH PRESSURE

be noted that the general shape of the curve is retained throughout this pressure range although it differs slightly from the typical curve at lower cell pressures (FIGURE VI.11). It is also quite unlike that for any other normally consolidated soil known to the author and may possibly be unique to this soil and the test pressures. It resembles the stress-strain curves characteristic of steel in that it shows definite yield and ultimate stresses.

While the curves enjoy a common initial tangent modulus, the "yield" strain increases from one to two per cent, and the ratio of maximum deviator stress to "yield" stress decreases from about 1.47 to 1.15 as the confining pressure is increased from 225 p.s.i. to 1500 p.s.i. . The strain at maximum deviator stress varies from 10 to 12 per cent and does not appear to be influenced by high cell pressures. On the other hand a typical stress-strain curve for tests at low cell pressures shows that the deviator stress is still increasing at 20 per cent strain at which point it is approximately twice the "yield" stress. This difference is borne out by the appearance of the specimens at failure. Whereas they were generally barrel-shaped at 30 and 60 p.s.i., a faint shear plane was evident in the high pressure specimens. The Rendulic diagram

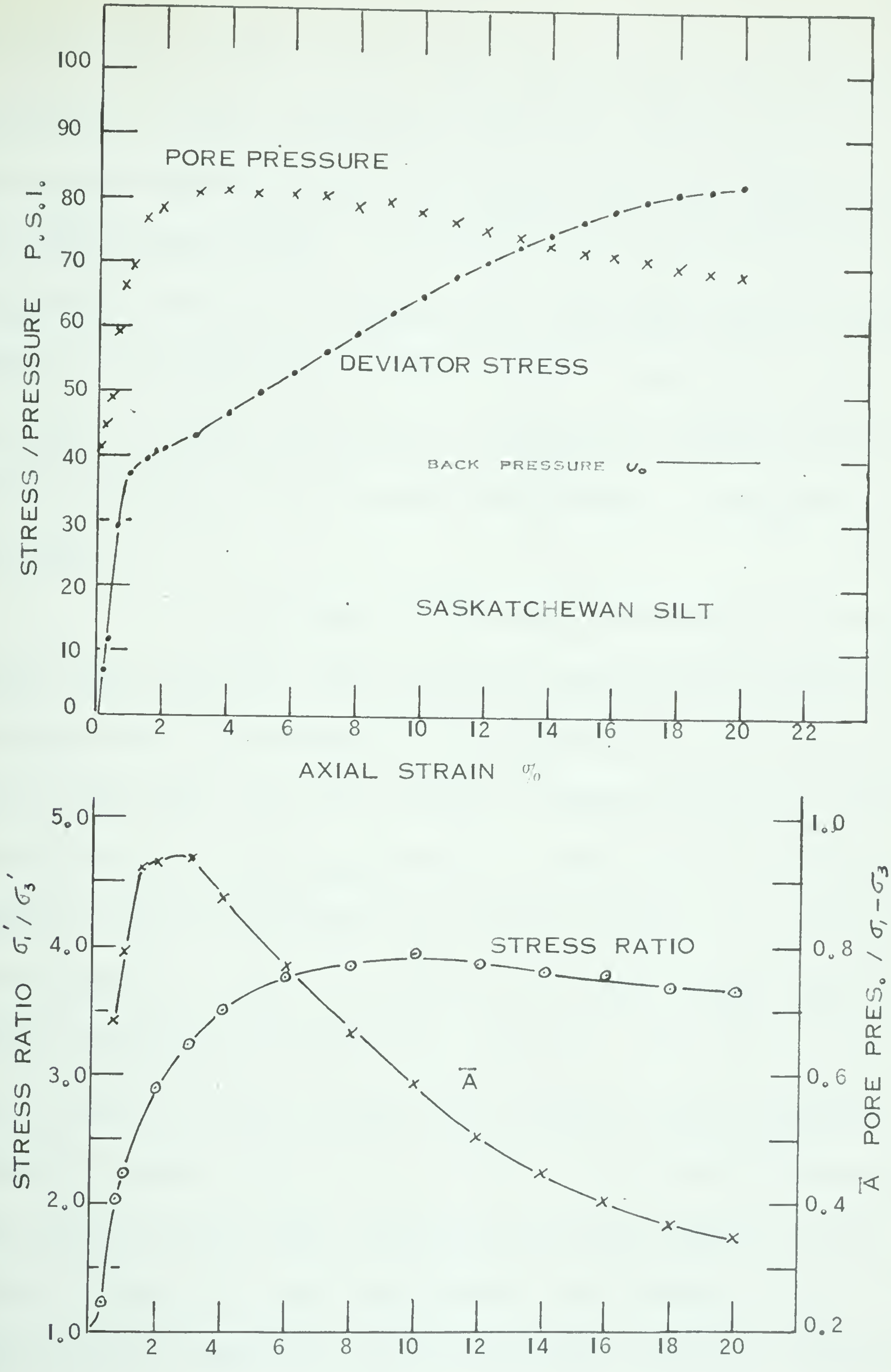

FIGURE VI.II STRESS - STRAIN CHARACTERISTICS, LOW PRESSURE

stress paths (FIGURE VI.12) also illustrate this difference in behavior. In this diagram the effective stress paths followed during the shearing phase are plotted using the axial stress, $\sqrt{2}$ radial stress plane. A very distinct hook at about 10% strain, suggesting failure, is apparent for the high pressure specimens. There is no suggestion of such a hook in the stress path of specimen 7 tested at 60 p.s.i. This path coincides with the failure envelope from five per cent strain to completion of the test.

In contrast to the above, Hirshfeld and Poulos (1963) have reported the results of consolidated-drained tests conducted on undisturbed silt at pressures up to 570 p.s.i. which show that the axial strain at maximum deviator stress increases with confining pressure. Observations of volumetric changes with strain showed an initial decrease in volume for all specimens but that, at failure, the silt became less dilatant as the confining pressure was increased. This same conclusion may be drawn from the values of $\bar{A}$ (ratio of pore pressure to deviator stress) observed at failure in the Saskatchewan Silt. The increase in $\bar{A}_f$ with confining pressure appears to be nearly linear on a semi-logarithmic plot (FIGURE VI.13). This would seem to indicate that the behavior of silt at high con-

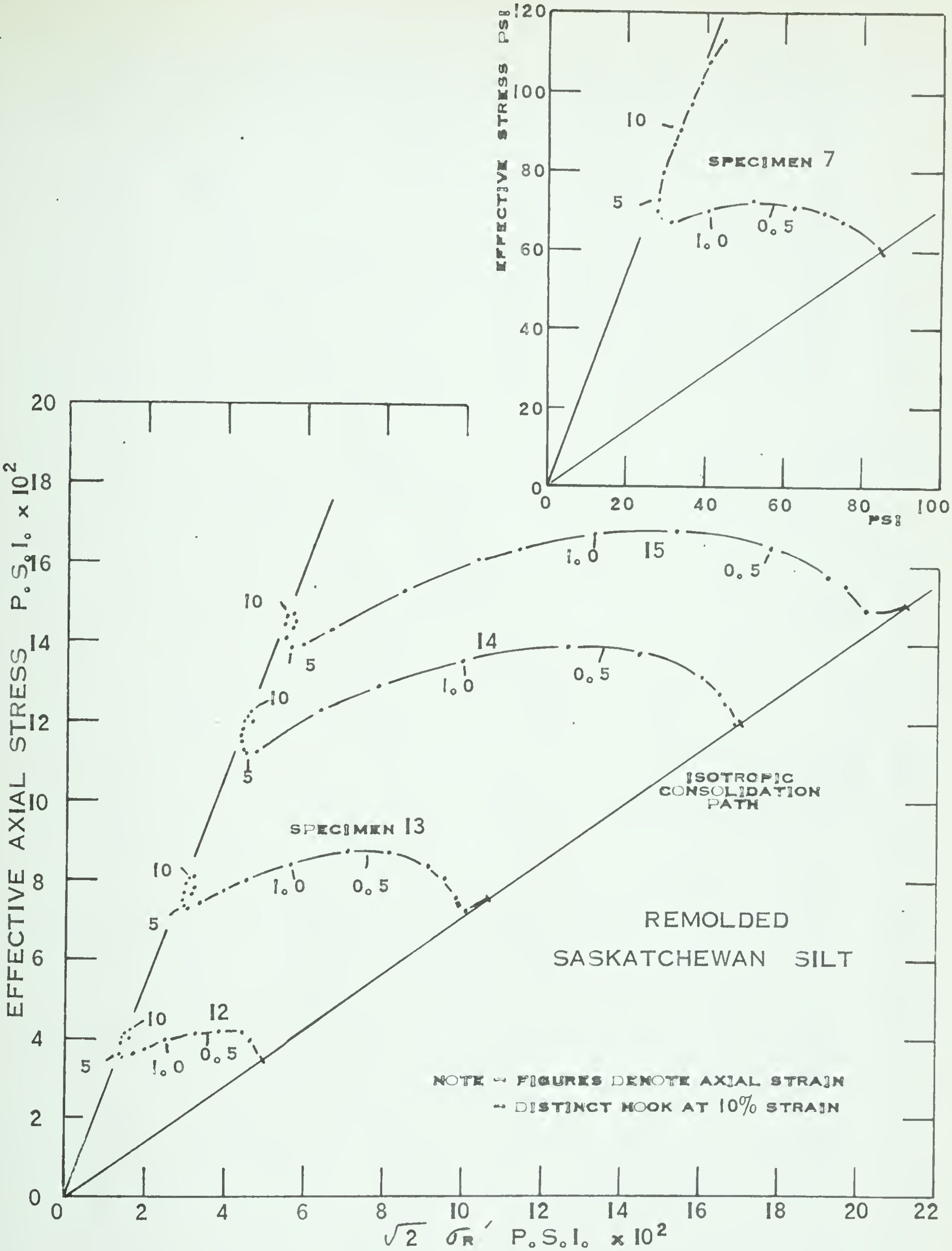

FIGURE VI.12 EFFECTIVE STRESS PATHS

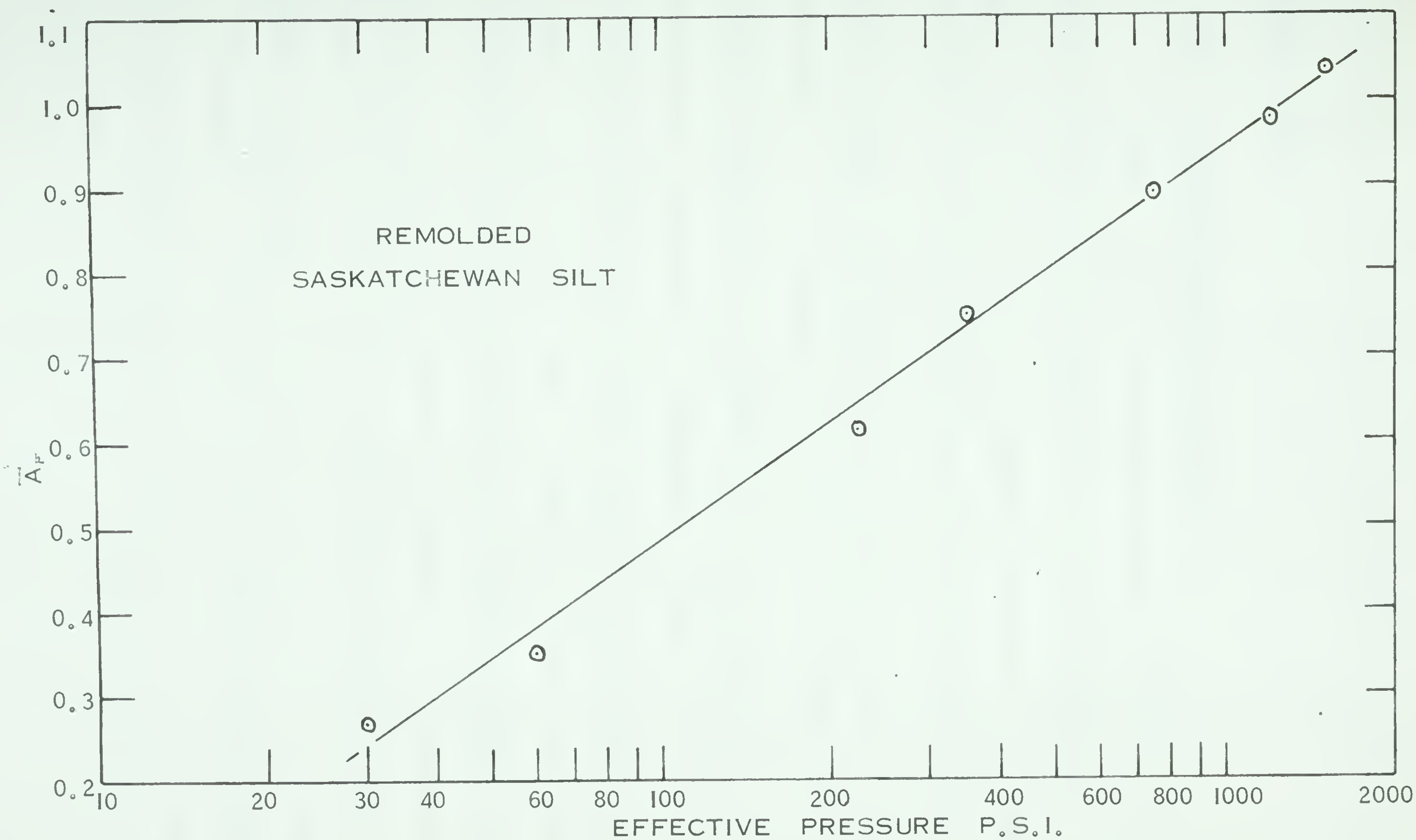

FIGURE VI.13 INDUCED PORE PRESSURE AT FAILURE vs CELL PRESSURE

fining pressures is opposite to that normally assumed but may not be indicative of conditions on the failure plane (Shockley and Ahlvin, 1960).

(e) Strength Characteristics

Mohr rupture lines are shown in FIGURE VI.14 (effective stresses) and FIGURE VI.15 (total stresses). While additional tests to confirm the results would be desirable, the available evidence indicates that the envelope passes through the origin and rises at an effective angle of shearing resistance of 34 degrees. It will be recalled that the same slope is exhibited in FIGURE VI.6. No reduction in ϕ' with increasing pressure is indicated. There is reason to believe that the effective stress circle for specimen 15 is plotted too far to the left. This is likely a manifestation of membrane leakage resulting in slightly higher pore pressures than would otherwise be expected. The suspicion is substantiated by the total stress Mohr diagram which shows the circle for specimen 15 within the envelope. It would appear, therefore, on the basis of the limited data shown that the Mohr-Coulomb failure criteria is valid for Saskatchewan Silt at pressure up to 1500 p.s.i. . Since there is no evidence

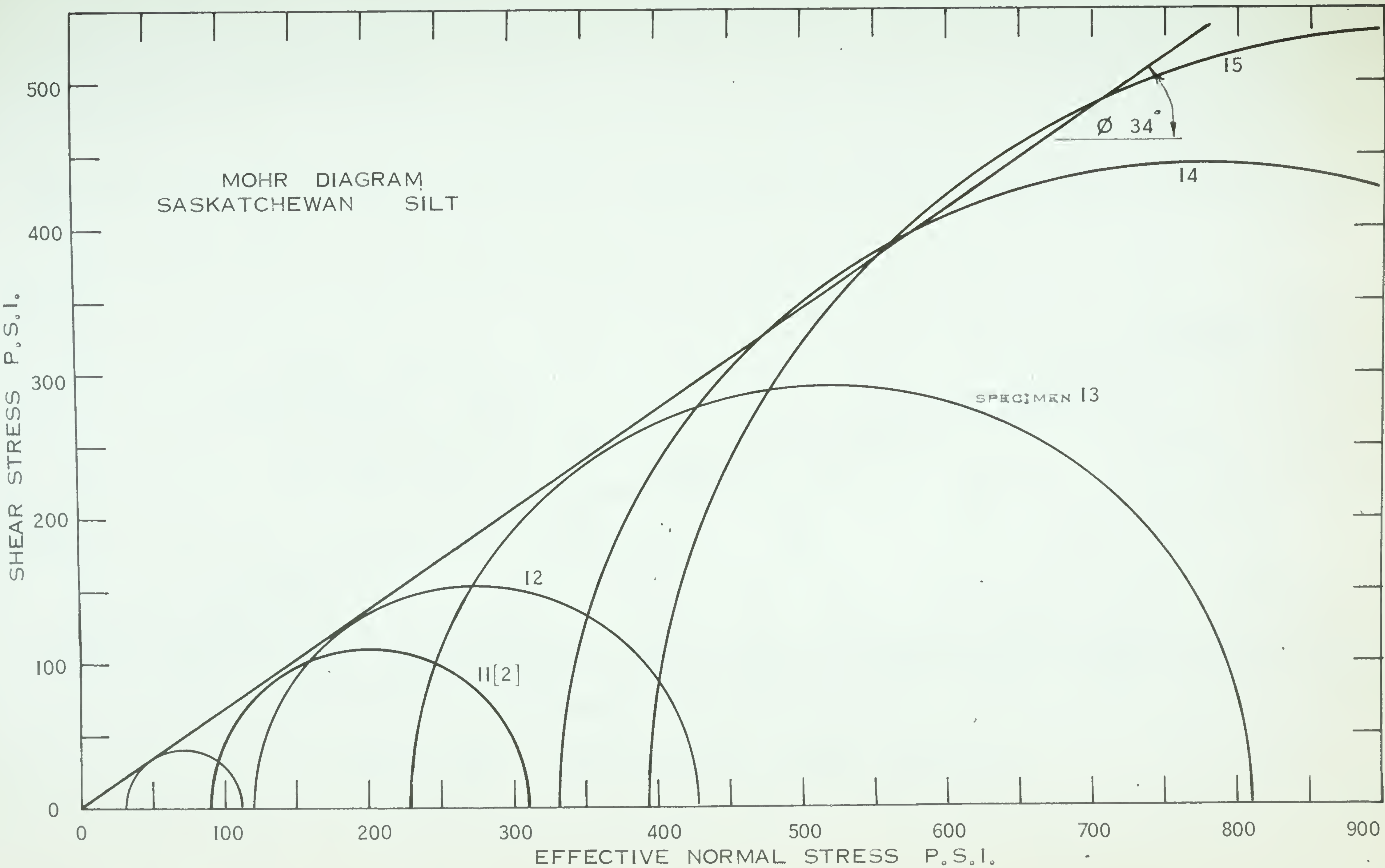

FIGURE VI.14 MOHR DIAGRAM – EFFECTIVE STRESSES

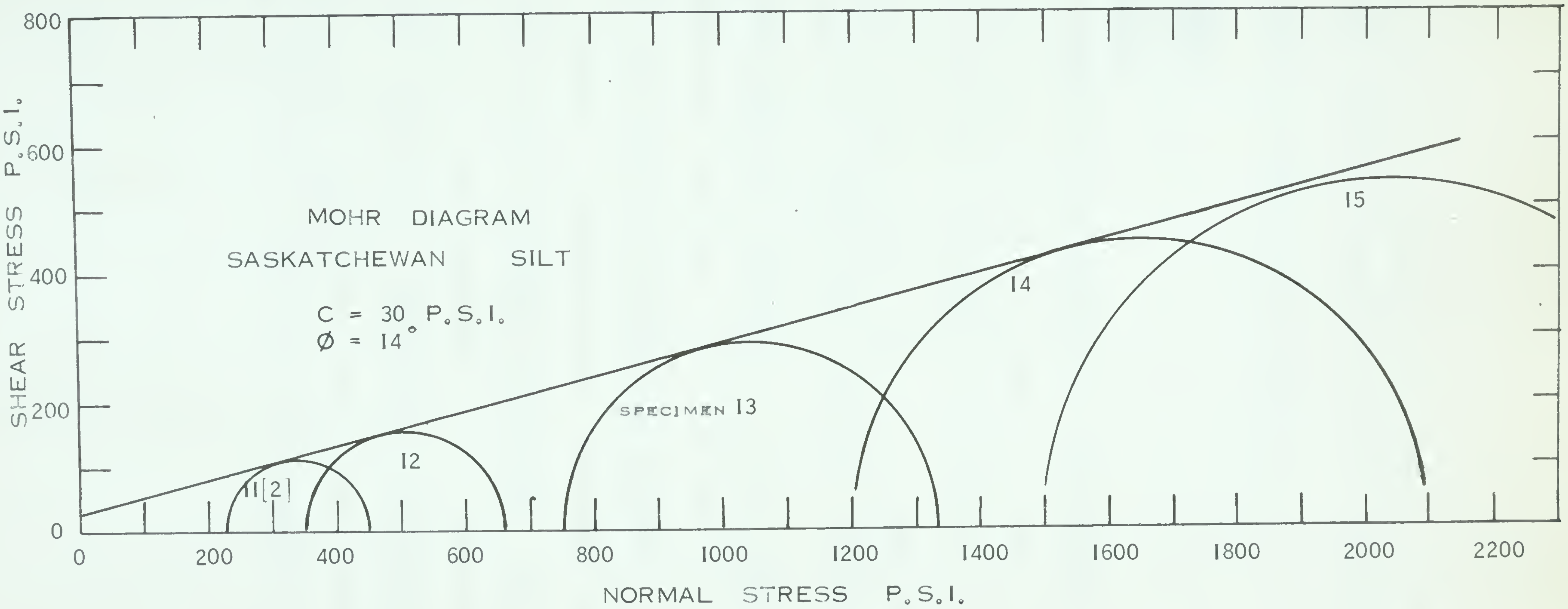

FIGURE VI.15 MOHR DIAGRAM – TOTAL STRESSES

of particle degradation or reduction in shearing resistance with pressure, there should be little problem in extrapolating strength from tests at normal cell pressures.

6.5 Bearpaw Shale

(a) General

As pointed out in Chapter I, the original aim of this investigation was to obtain information of the stress-strain characteristics of the shale at pressures up to the estimated preconsolidation load. This aim was defeated by the problem of membrane leakage and consequent inability to successfully consolidate the specimens. Eleven specimens were trimmed and mounted for testing. Four of these were reconsolidated at least once because leakage was apparent. The results of eight tests carried to completion are summarized in TABLE VI.3 and discussed in the following paragraphs. The same magnitude of possible errors reported for the Saskatchewan Silt apply to this discussion. Because of the leakage, rate of strain, and lack of comprehensive data, any conclusions drawn from this phase of the work must be regarded as tentative only.

TABLE VI.3

SUMMARY OF TEST DATA – BEARPAW SHALE

TEST DATA	SPECIMEN NUMBER							
	1	3	4	5	6	7*	10	11
DEPTH FEET	103	97	97	97	97	97	97	103
INITIAL MOISTURE CONTENT %	23.12	26.21	26.27	27.13	26.42	26.91	26.80	23.54
INITIAL VOID RATIO	.63	.74	.72	.72	.71	.73	.735	.66
INITIAL DEGREE OF SATURATION %	99	96	98.8	100	100.4	99.1	98.4	96.1
CELL PRESSURE PSI.	80	80	80	80	80	250	120	120
VOLUME CHANGE C.C.	-1.64	-0.27	+0.09	+0.25	-0.64	-2.76	-1.35	-2.32
T_{100} MINUTES	240	1500	700	800	330	–	250	500
BACK PRESSURE PSI.	20	20	20	20	20	40	20	20
PORE PRESSURE REACTION B	.28	.82	.84	.81	.23	1.15	.34	.13
RATE OF STRAIN [NOM.] HOURS FOR 1%	2.5	0.25	10.5	20.4	1.0	40	40	40
RATE OF STRAIN [ACTUAL] HOURS FOR 1%	10	1.0	22	48	4.25	78	75	84
AXIAL STRAIN AT FAILURE %	1.8	2.8	1.85	2.4	2.6	0.7	1.5	1.9
PEAK DEVIATOR STRESS PSI.	208	258	214	220	247	201	177	245
RESIDUAL DEVIATOR STRENGTH PSI.	138.6	132.7	154.6	111.9	102.3	82.4	84.8	146.6
$\bar{A}_F$ [PEAK DEVIATOR]	.185	.194	.23	.18	.198	1.04	.375	.155
VOID RATIO AT FAILURE	.66	.75	.725	.75	.744	.778	.736	.647
FINAL DEGREE OF SATURATION %	102	102.4	101.8	103.6	102.8	101.5	101.2	100.6
ANGLE OF SHEAR PLANE – DEGREES	66	58	64	59	55	60	32	55

* SIGNIFICANT MEMBRANE LEAKAGE

(b) Consolidation Characteristics and Permeability

The first five specimens were consolidated at the same cell pressure to permit a comparison of pore pressure with rate of strain. Eighty pounds per square inch was a poor choice as it corresponds closely with the reported swelling pressure of the shale (Ringheim, 1964). A typical time curve shows initially a decrease in volume of about 0.5 c.c. followed by an increase in volume of the same magnitude and then a further decrease. The initial reduction was generally completed within 10 to 20 minutes and is considered to represent the egress of extraneous water within the system. The second period of apparent volume reduction, (not observed in all cases) is likely indicative of membrane leakage.

It should be pointed out that the volume changes noted in TABLE VI.3 have been "corrected" by neglecting the volume expelled during the first six seconds. Nevertheless, a discrepancy between the observed change in burette reading and the difference between initial and final wet weights exists. This discrepancy varies from 1.5 to 3.5 c.c. and is partly a result of membrane leakage. Of equal significance in explaining the anomaly is the dilatant nature of the shale. On release of the cell pressure at the end of a test, the

specimen has a desire to draw free water from the porous stone and pedestal. The drainage aids, in effect, act in reverse and transmit water from the pedestal to the surface of the specimen within minutes. Henkel and Sowa (1963) have found that the discrepancy between volume changes may be eliminated if the specimen is allowed to swell under a few p.s.i. at the end of the test. This procedure was not adopted for two reasons. To observe the volume of water imbibed a burette must be reconnected to one of the drainage ports. Since water must then be recirculated through the pedestal the actual volume taken up by the specimen is indeterminant. Further, the time required for equilibrium to be achieved could be excessively long. The first objection can be overcome by originally consolidating the specimen against a back pressure and noting the volume expelled by means of a twin volume change burette. This procedure should be adopted in future testing of the shale. The second objection, however, cannot be overcome and would likely make the rebound procedure impracticable, particularly for tests at the higher cell pressures.

An e-log p curve for the shale obtained from one dimensional consolidation to 110 tons/sq.ft. is shown at Appendix E. The coefficient of permeability, computed from the data on which this curve is based, is 5×10^{-10} cm./sec. at 120 p.s.i. . This corresponds to a coefficient of consolidation, c_v, of 2.2×10^{-4} $cm.^2$/sec. . From the theoretical time curve presented by Gibson and Henkel (1954) the coefficients of consolidation for radial and end drainage corresponding to values of t_{100} of 250 and 500 minutes are 1.6 and 0.8×10^{-4} $cm.^2$/sec. . If an average value of 800 minutes is assumed from the P.F.R.A. data at Appendix E. the coefficient at 100 p.s.i. is 0.5×10^{-4} $cm.^2$/sec. . These values are therefore of the same order for both one dimensional and three dimensional consolidation.

At any given pressure the coefficient of consolidation varies directly as the coefficient of permeability. If the value of k is as low as 10^{-14} cm./sec., as some authorities believe, a conservative estimate of the time

required to consolidate a single 1.5 inch diameter specimen with peripheral drainage aids would be six years. Obviously the consolidated-undrained test, in its present form, would be rendered useless under such circumstances.

(c) Pore Pressure Reaction

All but one of the pore pressure reactions were poor. The exception, specimen 7, was high because of membrane leakage. Of the remainder, the best reaction was 84 per cent even though the buildup of pore pressure was allowed to continue for 40 minutes.

The factors discussed in regard to the reaction of Saskatchewan Silt apply equally to the shale although the shale is slightly more compressible under one dimensional drained conditions. While the modulus of elasticity reported for Bearpaw Shale is only 16,500 p.s.i. (Ringheim, 1964), values ranging from 20,000 p.s.i. to 140,000 p.s.i. have been reported by Knight (1962) for the Pierre Shale and similar Missouri bedrock formations. An estimated initial tangent modulus taken from the stress-strain curves of this investigation is 25,000 p.s.i. . Obviously, some other explanation is required.

The clay fraction of the shale is predominantly sodium montmorillonite, a clay mineral complex which has associated with it comparatively large adsorbed water layers. It is known that at natural water contents in the order of 25 per cent, a very high percentage of the total moisture exists in the form of adsorbed water hulls. Although pressure may be transmitted through the adsorbed moisture, its higher viscosity serves to reduce the speed of transmission and increase the difficulty of recording accurate pore pressures. Dahlman (1965) observed reactions for homionic sodium montmorillonite as low as 14 per cent. Since his specimens were remolded at an initial void ratio of approximately two and were therefore highly compressible, it is apparent that the very low coefficient of permeability of this material must be a factor. A rigid structure is not the dominant cause of poor reactions in the shale.

(d) Stress-Strain Characteristics

One of the most important factors which influences the strength of cohesive soil is the rate at which it is strained, Casagrande and Wilson (1951) have shown that the undrained strength of undisturbed Bearpaw Shale is reduced by

20 per cent under conditions of extended load duration.

Of more immediate concern, however, is the influence of rate of strain on pore pressure equalization. Blight (1963) has produced a chart based on experimental data which provides the test duration giving 95 per cent equalization of pore pressure for known values of c_v and length of drainage path. If c_v is assumed to be 5×10^{-5} cm^2/sec., Blight's chart shows a test duration of 4.5 hours for a 1.5 inch diameter specimen with all-round drains. To obtain 99 per cent equalization this time would increase to about seven hours (Penman, 1961). The meaning of the term "test duration" used on the chart depends upon the object of the test. If the object is to measure shearing resistance only, the duration of test may be taken as the time to failure. If complete information on the stress path is required, the duration is the period between the start of the test and the first significant stress measurement (Blight, 1963).

Specimens 1 to 6 inclusive were strained at different rates in an attempt to relate $\bar{A}_f$ with the time to failure. Because of the wide natural variation in strength this attempt was unsuccessful. It is interesting to note, however, that in all cases the pore pressure was decreasing

at maximum deviator stress suggesting that the shale has dilatant tendencies. While this would not normally be considered an unusual observation of such a dense clay structure, it has not been previously recorded in the testing of Western Canadian clay shales (Hardy, 1965). The dilatant nature was also apparent from the distribution of moisture within the specimen at the end of the test. The moisture content in the immediate vicinity of the failure plane was visually higher than that in the remainder of the specimen. That is, the failure zones were soft and remolded within a thickness of about 0.10 inches. This is not evident, in every case, from the data of TABLE VI.4 because it was not feasible to obtain accurate moisture contents from sufficiently thin slices.

TABLE VI.4

MOISTURE CONTENT VARIATION AT END OF TEST
BEARPAW SHALE SPECIMENS

Specimen	Moisture Content - per cent		
	Top	Failure Zone	Bottom
1	22.51	22.52	23.47
3	25.98	28.21	27.92
4	27.17	26.90	27.90
5	27.29	27.87	26.82
6	27.37	29.04	26.18
7	29.36	30.54	27.24
10	26.79	26.67	28.22
11	23.28	23.60	24.00

Specimens 7, 10 and 11 were strained at the slowest possible rate (nominally 1.3×10^{-5} inches per minute). This is equivalent to one per cent in 40 hours. Because of the relative strengths of specimen and proving ring, the actual time required for one per cent strain averaged 80 hours. Times to failure were, therefore, about five days.

The most significant features of the stress-strain data are the very low failure strains and immediate reduction to an apparent residual value. Stresses beyond failure were calculated on the basis of the projected area of contact along the failure surface. In effect this area is the horizontal projection of a traversed ellipse. This method tends to increase the residual strength by about ten per cent (at eight per cent strain) above that which would be calculated with the usual area corrections. The ratio of residual to peak strength ranged from 0.39 to 0.67 with an average of 0.55.

The stress paths illustrated in FIGURE VI.16 are characteristic of overconsolidated soils (Henkel and Sowa, 1963) and are distinctly different from those plotted for the silt specimens. The influence of membrane leakage on specimen 7 is very apparent when stresses are plotted in this manner.

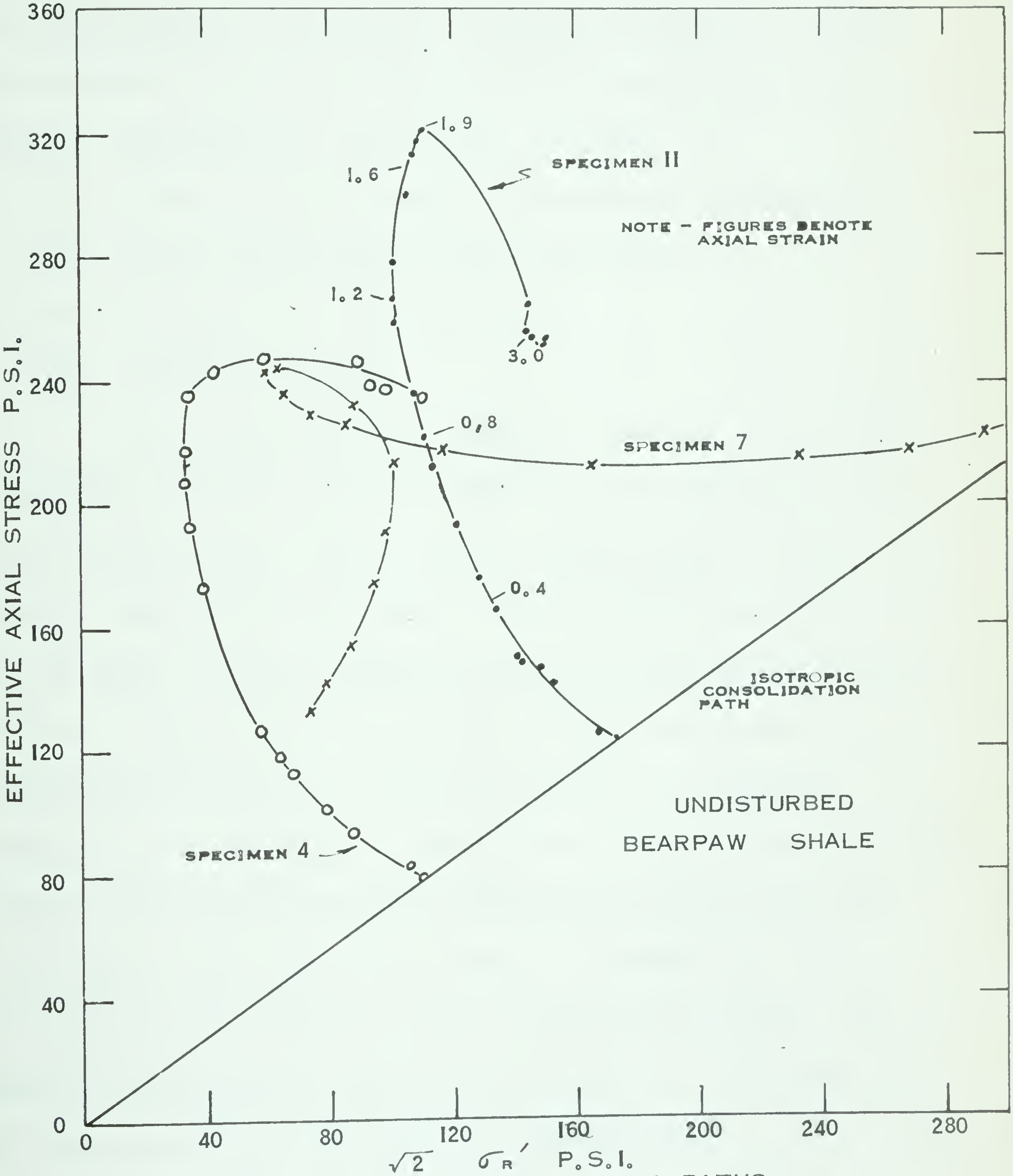

FIGURE VI.16 EFFECTIVE STRESS PATHS

(e) Strength Characteristics

It is fairly evident from FIGURES VI.17 and VI.18 that it is virtually impossible to estimate the shale's shearing resistance with any confidence on the basis of the data obtained. The principal reasons for this are:

(i) wide natural variation in compressive strength,

(ii) the limited range of cell pressures,

(iii) variation in rate of strain,

(iv) membrane leakage.

If a zero effective cohesion intercept is assumed, the apparent effective angle of shearing resistance may range from 37 to 55 degrees (lines a and b, FIGURE VI.17). These values are unrealistically high. A more probable envelope is shown as line c which is plotted tangent to two average circles at 80 p.s.i. and one at 120 p.s.i. . It gives effective strength parameters of $c' = 80$ p.s.i. and $\phi' = 12$ degrees and may be quite realistic in terms of the Krey-Tiedeman criteria. If the preconsolidation pressure is assumed to be in the order of 150 tons/sq. ft., the stress ranges of FIGURES VI.17 and VI.18 represent only ten per cent of the entire picture. The parameters noted may be confirmed and brought into perspective once the envelope is established to this pressure.

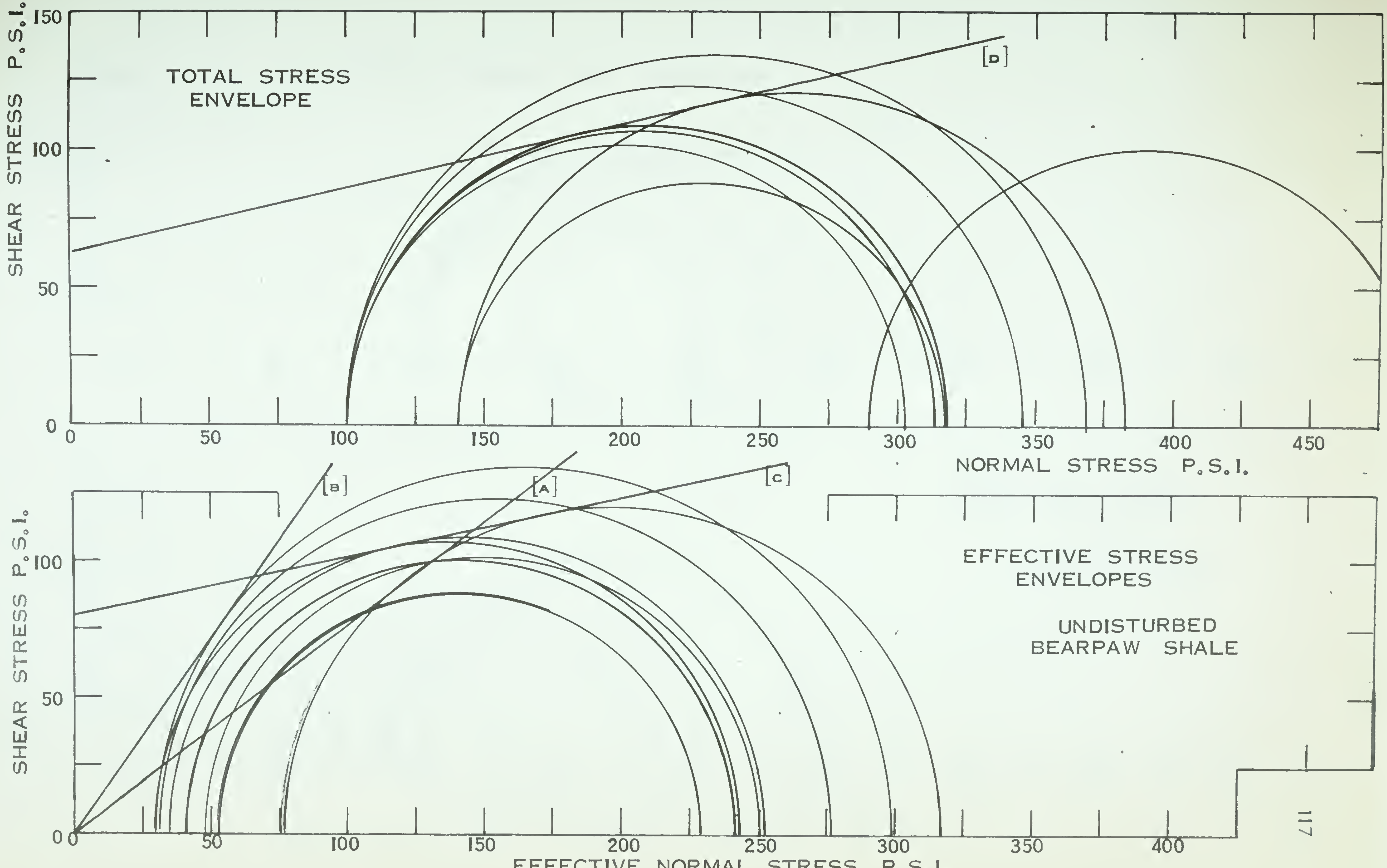

FIGURE VI.17 MOHR DIAGRAMS – MAXIMUM DEVIATOR STRESSES

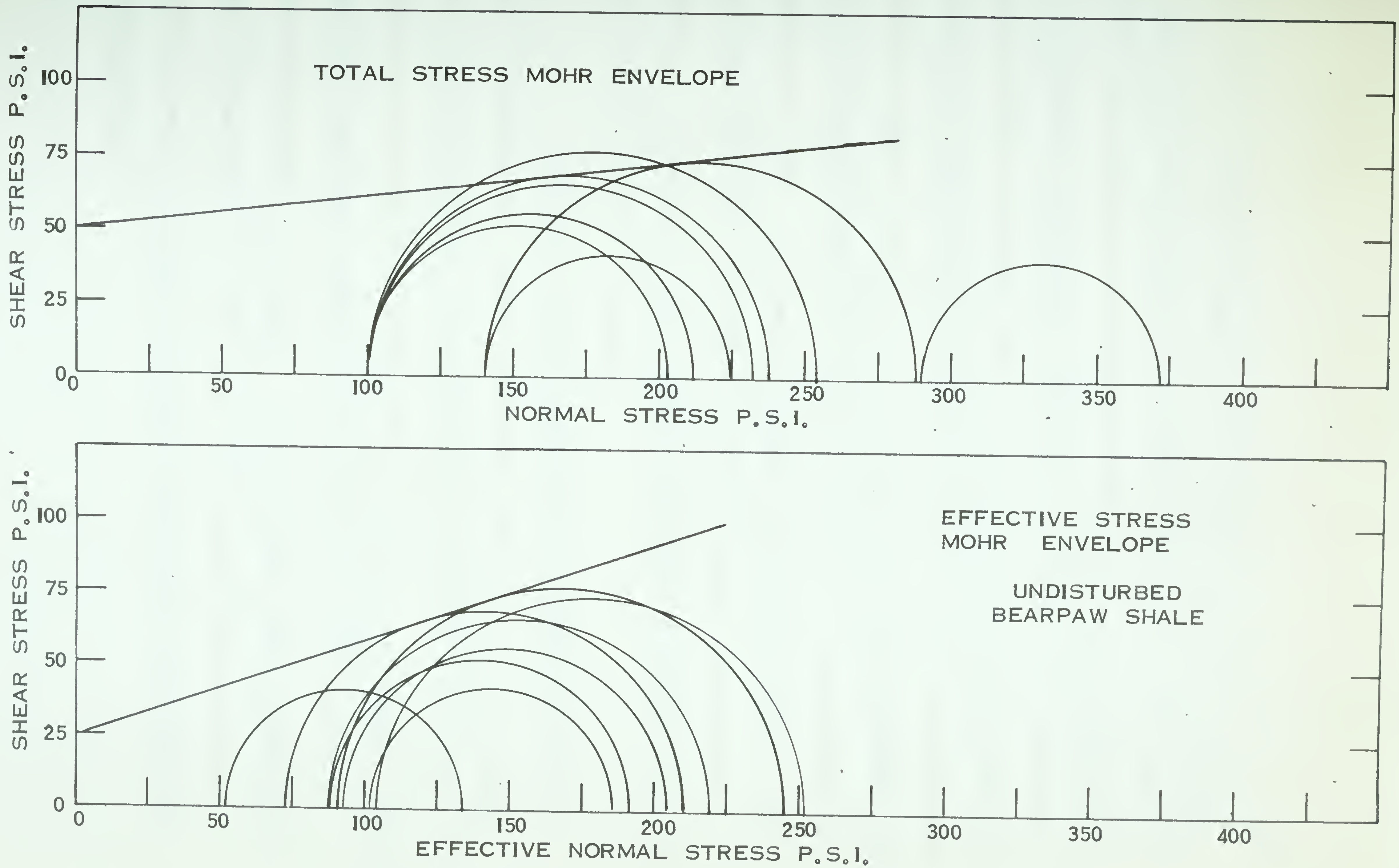

FIGURE VI.18 MOHR DIAGRAMS — RESIDUAL STRENGTHS

The inclination and location of line d are not significantly different from those of line c. This suggests that the measured pore pressures have not really influenced the shearing resistance. However, to improve the situation appreciably, extremely low pore pressures at failure would be required. While such low values might be attained by greatly reduced rates of strain, it is likely that the sustained loading would also reduce the maximum deviator stress in a manner similar to that noted by Casagrande and Wilson (1951). The relative influence of reduced pore pressures and reduced deviator stresses on the overall shearing resistance would not be a simple matter to assess.

Residual strength envelopes (FIGURE VI.18) are seen to reduce the overall shearing resistance of the shale as would be expected. The indicated effective stress envelope suggests a 70 per cent reduction in cohesion intercept. However, parallel envelopes showing an even greater reduction could be drawn with equal confidence. It is evident that a great many more stress circles covering a wide range of pressures are required before a proper strength envelope for the shale can be established.

6.6 Summary

In this chapter the experimental data and results have been presented and discussed. It has been shown that the standard membranes and O-Ring bindings permitted significant leakage of the cell water into the specimens. Most of this leakage appears to have occurred past the bindings and may have been partially caused by the presence of rubber "dams". A new rubber jacket has been designed to combat the problem.

Pressure transducers coupled to a strain indicator have been shown to be an accurate and sensitive means of monitoring pore pressures over extended periods of time. They constitute the first steps towards a fully automated data processing system.

The effective shearing resistance of remolded Saskatchewan Silt has been obtained at pressures up to 1500 p.s.i. . Poor pore pressure reactions were observed and partially attributed to the rigidity of the soil structure at high cell pressures. Stress-strain curves which illustrate distinct yield and ultimate stresses at high pressures have been shown. The effective angle of shearing resistance appears to be 34 degrees.

Very low pore pressure reactions were also observed for the Bearpaw Shale and attributed to the high percentage of adsorbed water and consequent low permeability. Peak deviator stresses occurred at very low axial strains and varied widely between specimens of essentially the same moisture content. Apparent residual stresses averaging 55 per cent of the peak values were also well established at fairly low strains. The data were inconclusive insofar as shearing resistance is concerned.

CHAPTER VII

CONCLUSIONS AND RECOMMENDATIONS

7.1 General

A number of slope failures throughout Western Canada have shown the Upper Cretaceous clay shales to be troublesome formations under certain circumstances. Instability is not a general condition but is dependent on many geologic factors which are not completely understood. This is particularly true of the Bearpaw formation.

A review of the literature has revealed four approaches which have been taken to assess the shearing resistance of overconsolidated clay shales. These concepts modify either the traditional strength theories or the laboratory strength parameters, or abandon both in favour of field observations.

The purpose of this investigation has been to extend the application of the triaxial compression test to high pressures with a view to obtaining parameters of shearing resistance for the Bearpaw Shale which reflect the true field

condition. The program included tests to determine the adequacy of rubber membranes and bindings, and tests to develop techniques for the use of high pressure triaxial compression equipment and electrical pressure transducers.

7.2 Conclusions

The following conclusions are based on the membrane tests and concolidated undrained tests on Saskatchewan Silt and Bearpaw Shale.

1. Membrane leakage presents a severe problem in high pressure, long duration undrained compression testing. No relationship between observed rate of leakage and applied pressure or combination of membranes and bindings was established. The leakage is believed to occur primarily past the O-Ring seals and may be caused, for the most part, by the rubber "dams" which are normally used.
2. The electrical pressure transducer is a simple and accurate means of measuring pore pressures. It is particularly suitable for long duration tests. The steel adaptor manufactured to conjoin the transducer and triaxial compression cell functions

satisfactorily.

3. The silt exhibited poor pore pressure reactions, particularly at the higher cell pressures. This may be attributed partially to the relatively low compressibility of the soil structure. The basic assumption that the compressibility of water is infinitely small compared to that of the soil structure is seriously in error.
4. Bearpaw Shale also exhibits low values of the coefficient B. In this case the poor reaction may be due primarily to the slow transmission of pressure through adsorbed water layers and consequent low coefficient of permeability.
5. Deviator stress-strain curves for the silt exhibit definite yield and ultimate values and appear to be unique. The specimens yield at one to two per cent strain and achieve ultimate strength at 10 to 12 per cent strain under high confining pressures. The constrained initial tangent modulus is 100,000 p.s.i. . The pore pressure coefficient, $\bar{A}$, at failure, increases with increasing cell pressure. The effective angle of shearing resistance is 34 degrees throughout the

range of test pressures.

6. Deviator stress-strain curves for the shale peak at axial strains of one to three per cent and drop immediately to a residual value which averages 55 per cent of the peak stress. The shale exhibits dilatant tendencies at failure.
7. There are insufficient data on which to base a firm conclusion concerning the influence of the rate of strain on the accuracy of pore pressure measurements.
8. The overall shearing resistance of the shale has not been sufficiently well defined to shed any light on the problem set out in Chapter I.

7.3 Recommendations

The following recommendations are made for future research in this field.

1. A continued effort should be made to determine the cause of leakage of cell water into the test specimen. The use of a chemical or color tracer may be helpful. In particular the effect of dispensing with rubber "dams" and greasing the pedestal and

top cap should be investigated. It may well prove that such a simple and inexpensive correction of present procedures will rectify the situation and result in more accurate data from routine tests.

2. Techniques for employing the proposed rubber jacket on shale specimens must be developed; particularly to ensure saturation of the system. The jacket's influence on recorded strength must be evaluated.
3. Before any further testing of the shale is conducted, the data recording system should be automated as far as possible. This system should employ internal load cells in lieu of the present proving rings.
4. A program to determine the appropriate rate for straining shale specimens to permit accurate measurements of pore pressure in the failure zone must be pursued. It would not appear practicable to insert a probe near the failure surface. Therefore, the rate of strain must be low enough that the pore pressure measured at the base of the specimen is equal to that existing in the zone of failure.
5. It is recommended that the strength envelope for the undisturbed material be extended to 2000 p.s.i. .

It would also appear prudent to consolidate specimens at 2000 p.s.i. and then rebound them to "in situ" pressures before testing. They should be consolidated against a back pressure.

6. The feasibility of preparing specimens of remolded shale for the high pressure cell should be investigated. If this is feasible, data of the shale's behavior may be obtained under more closely controlled conditions.

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Appendix A

HIGH PRESSURE APPARATUS

APPENDIX A

HIGH PRESSURE APPARATUS

A.1 General

In order to permit testing of the shale at pressures up to the estimated preconsolidation load, a system capable of sustained working pressures to 2000 p.s.i. was purchased from Wykeham Farrance Engineering Ltd., England. This apparatus consists essentially of a combined constant pressure and pore pressure system plus two cells with auxiliary equipment.

A.2 The Cell

The cell is shown in FIGURE A.1 . It is constructed of steel and brass throughout except for three windows. It stands 12 inches high, has a base diameter of eight inches and weighs about 40 pounds empty. The piston is one inch in diameter except at the bottom where it is flared or barbed. This shape serves two functions. It allows the piston to seat properly in a rounded conical recess in the loading cap

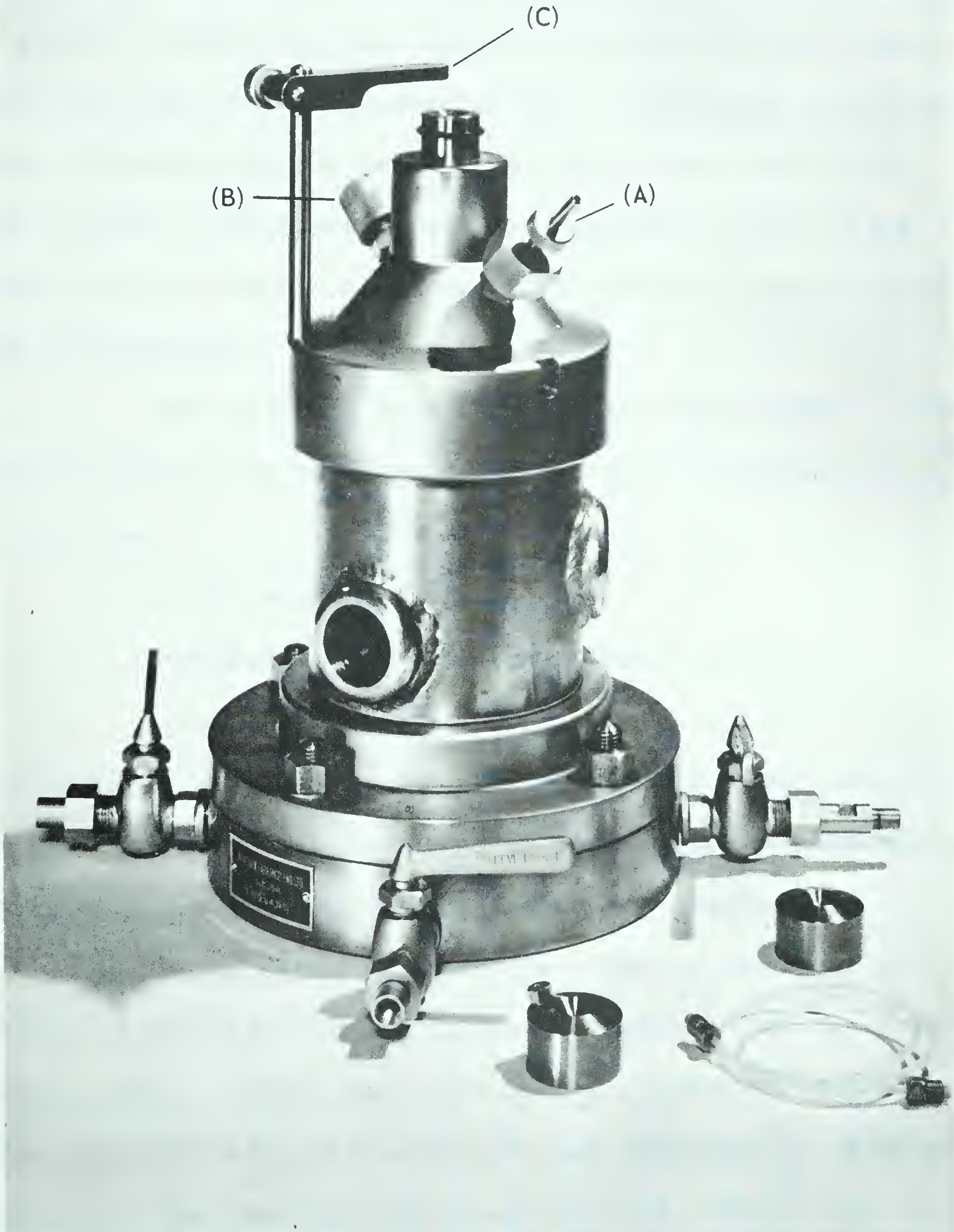


FIGURE A–1. HIGH PRESSURE TRIAXIAL CELL

without the necessity of using a ball bearing. It also retains the piston within the cell under pressure without the use of a proving ring or other external aid. The piston is prevented from dropping into the cell by an O-Ring which encircles the top end. The pressure seal is provided by one O-Ring embedded in a closely machined, fixed bushing through which the piston moves.

The cell base is equipped with four Klinger valves which serve the same functions as in low pressure cells, i.e.;

1 cell water connection

2 pedestal drainage connections

1 top cap drainage connection.

For purposes of measuring pore pressure during the test one of the Klinger valves connected to a pedestal drainage port is removed and replaced by the transducer adaptor.

The pedestal is removable. This permits the exchange of pedestals with and without embedded high-air-entry ceramic discs. While the cells used are equipped with 1.5 inch diameter pedestals only, two inch pedestals may also be employed. The drainage ports which extend through the

pedestal and base are protected by small rubber 0-Rings to seal off chamber fluid. A six inch rubber 0-Ring is fitted against the circumference of the recessed central portion of the base. This 0-Ring effects the seal between the base and the upper portion of the cell.

The upper body of the cell is fixed to the base by means of six threaded bolts protruding from the base through a flange on the upper body, steel washers, and hexagonal nuts. The principal features of the upper portion of the cell include the windows, an air release valve (A), an oil injection valve (B), and an arm on which to rest the strain dial (C). The cell was used at pressures up to 2000 p.s.i. and performed satisfactorily except for some leakage from the upper drainage Klinger valve at pressures in excess of 1000 p.s.i. It is considered that the performance of the cell might be improved by replacing the Klinger valves by 0.25 inch Circle Seal miniature shut off valves.

The constant pressure apparatus is shown schematically at FIGURE A.2 . It consists essentially of an oil over water system in which the pressure is maintained by weights suspended from a needle ram floating in a constant pressure pot filled with castor oil. As the pressure in the system

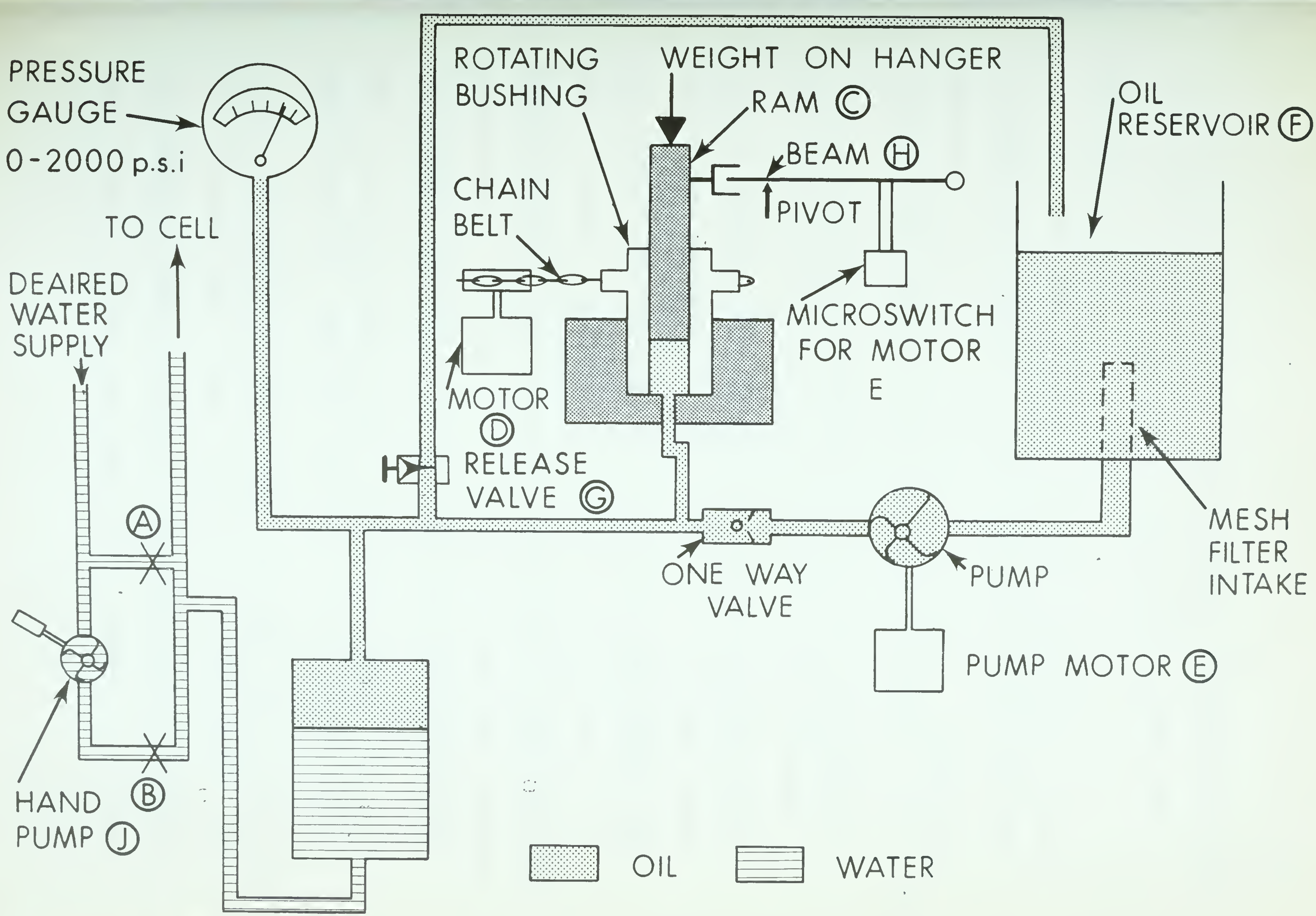


FIGURE A. 2 SCHEMATIC DIAGRAM - CONSTANT PRESSURE EQUIPMENT

drops because of leakage and/or specimen consolidation, this piston sinks and activates the pump which restores the pressure.

The dead weight of the piston and hanger imparts a pressure of 150 p.s.i. For operating at cell pressures above 150 p.s.i. the procedure is as follows. Weights are placed on the hanger to give the required pressure, five pounds being approximately equivalent to a pressure of 100 p.s.i. The cell is allowed to fill with deaired water through valve A (FIGURE A.2). When water emerges from the air release valve at the top of the cell, this valve and valve A are closed. The pressure in the cell is then raised, with valve B open, by means of the hand pump until the ram (C) starts to rise. The pressure in the system is then approximately that required and the ram is at or near the bottom of its travel. At this point the two motors are switched on. Motor D continuously rotates the bushing around the ram to reduce friction. The pump motor (E) is on because the ram is near the bottom of its travel. The pump takes castor oil from the reservoir (F) and forces it into the system. As the pressure rises to slightly above the required pressure the ram rises, tilting the beam (H) and thereby activating

the microswitch which cuts out the pump motor. During a test there are four effects that alter the pressure in the system. These are:

(a) change in volume of the test specimen

(b) entrance of the loading piston into the cell

(c) leakage

(d) expansion of the system with pressure.

If the combined effect of these factors is to reduce the pressure then the hanger pressure exceeds the oil pressure and the ram (C) descends. The Beam (H) is tilted, activating the microswitch to start the pump motor whereupon the desired pressure is restored and so on. Once started the operation is completely automatic and a constant pressure is maintained within $\pm$ 0.5 per cent. The sensitivity is governed by freedom of movement of the ram in the balancing cylinder and the switches operating the pump. The pumping duration is about six seconds and ideally the pump should not operate more frequently than once every five minutes.

If the net effect of the above mentioned factors is an increase in pressure, say by the rapid entrance of the loading piston into the cell, oil must be bled from the system

manually by cracking the needle valve (G). At the completion of a test the pressure must be reduced slowly to avoid damage to the ram. The needle valve, which is normally closed, permits good control of pressure reduction.

Appendix B

PRESSURE TRANSDUCERS

APPENDIX B

PRESSURE TRANSDUCERS

B.1 History

The electrical pressure transducer is a relatively simple device which converts pressure on a diaphragm into an electrical signal which in turn is recorded by an indicator. Although only recently applied to the measurement of pore pressures in the laboratory, the principle is not new. Piezometers which employ the diaphragm principle have been used for many years. One of the earliest diaphragm piezometers which made direct use of electrical response for measuring pressures was designed by Terzaghi (1943). Platema (1953) followed with a piezometer design in which strain gauges were cemented directly to the reverse side of the diaphragm. With such an arrangement it is a relatively simple matter to correlate the pressure on the diaphragm with the strain indicator readings recorded.

This principle appears to have been first utilized for a laboratory device by the United States Bureau of

Reclamation and later by Whitman, Richardson and Healy (1961), and Govinda Rao et al (1961). Rao describes a pressure cell which consists of a metal diaphragm to which is affixed an SR4 strain gauge and which is connected to a Bourdon pressure gauge. Under pressure the diaphragm deflects causing a response of the strain indicator. Air pressure on the reverse of the diaphragm is raised by a foot pump until the indicator is rebalanced and the pressure required is noted on the pressure gauge. The advantages of such a system are ease in deairing and simplicity of operation. The principle has now been refined to the extent that the diaphragm deflections are small enough to impose an essentially "no flow" condition on the system but, at the same time, are sufficient to permit accurate recording of the pressure. While the transducer probably represents the ultimate in pore pressure measuring systems today, it is nevertheless an expensive and delicate instrument and has been criticized on these grounds (Raymond, 1964).

B.2 Description and Specifications

Three transducers were used in the course of this investigation. All three were obtained from the Dynisco Co.

Ltd. and are identical except for the range of pressures they are intended to measure. They are designated as follows: APT25-1C, APT25-3C, APT25-2M. In this designation the A refers to "absolute" pressure, PT25 is the model number, and 1C, 3C and 2M refer to the pressure ranges 0 - 100 p.s.i.a., 0 - 300 p.s.i.a., and 0 - 2000 p.s.i.a. respectively. The instrument is a small enclosed steel cylinder 2 3/16 inches long by 3/4 inches in diameter. The steel sensing diaphragm is an integral part of the transducer body and is flush with the surface in contact with the fluid through which the pore pressures are transmitted. The diameter of the diaphragm is slightly smaller for the higher pressure instruments. The PT25 model is machined with an external thread and hexagon nut at the diaphragm end to facilitate mounting. A metal-backed rubber washer provides an effective seal when clamped between the body of the transducer and a flat mounting surface.

Electrically, the PT25 incorporates a miniature unbonded strain gauge in the configuration of a four active-arm resistive Wheatstone Bridge. It is this precision gauge which is responsible for the inherent accuracy, stability and

performance dependability of the transducer. The outer case effectively isolates and protects the strain gauge mechanism from mechanical shocks and external temperature changes. It is constructed so that distortion of the case during mounting does not adversely affect the sensing diaphragm or strain gauge mechanism provided the recommended mounting torque is not exceeded. The performance and electrical specifications are outlined in TABLE B.1 and simplified drawings of the transducer are at FIGURE B.1 .

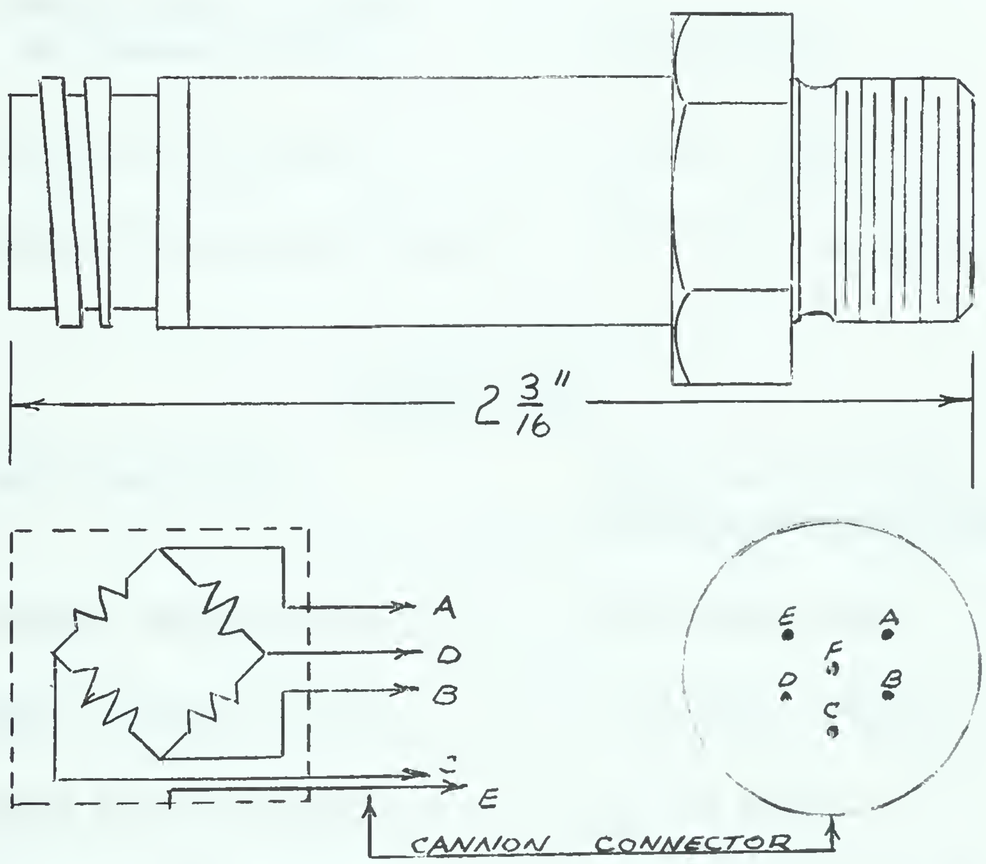


FIGURE B.1 Mechanical and Electrical Diagram

TABLE B.1

SPECIFICATIONS FOR PRESSURE TRANSDUCER MODEL PT25

Performance	
Max. Non-linearity	0.5%FS
Max. Hysteresis	0.25%FS
Repeatability	0.10%FC
Temperature Range	-65 to 300°F.
Temperature Drift	2%FS/100°F.
Temperature Effect on Sensitivity	1%FS/100°F.
Vibration Effect	0.007 psi/"G"
Rated Mounting Torque	100 inch pounds
Electrical	
Configuration	Four active-arm Wheatstone Bridge
Bridge Resistance	350 ohms nom.
Full Scale Output	4.0 mv/v (min.)
Zero Pres. Output	$\pm$ 1.0 mv/v
Excitation Voltage	6 volts ac or dc

B.3 Method of Use

For best results, transducers should be connected to the specimens by the shortest possible column of water. This may be accomplished in several ways. Burn (1964) describes a pressure block, designed to hold a transducer, which is connected to the triaxial cell by a length of tubing. This arrangement permits the block to be mounted on a stand and adjusted vertically to compensate final readings for the difference in pressure between the diaphragm and the specimen under test. A somewhat different arrangement, wherein the transducer-holding pressure block is sweated directly onto a cone connected at the base of the cell, is described by Richardson and Whitman (1963). Other variations have also been used successfully.

For purposes of this investigation a small cylindrical stainless steel adaptor (FIGURE B.2) was designed and constructed. The principal factors leading to this design were:

(a) compatibility with new high pressure cells,

(b) minimum column of water between specimen and sensing diaphragm,

(c) ease of deairing.

It is considered that the adaptors fulfil these requirements admirably. In operation, one of the Klinger valves controlling drainage from the pedestal is removed and replaced by the adaptor. It seats against a small rubber O-Ring at the base of the recess. The transducer is threaded into the adaptor taking care not to exceed the recommended torque. Deaired water may then be flushed through the pedestal and adaptor and egress through the miniature shut off valve. Suction may be applied to a line connected to this valve to ensure removal of air in the system.

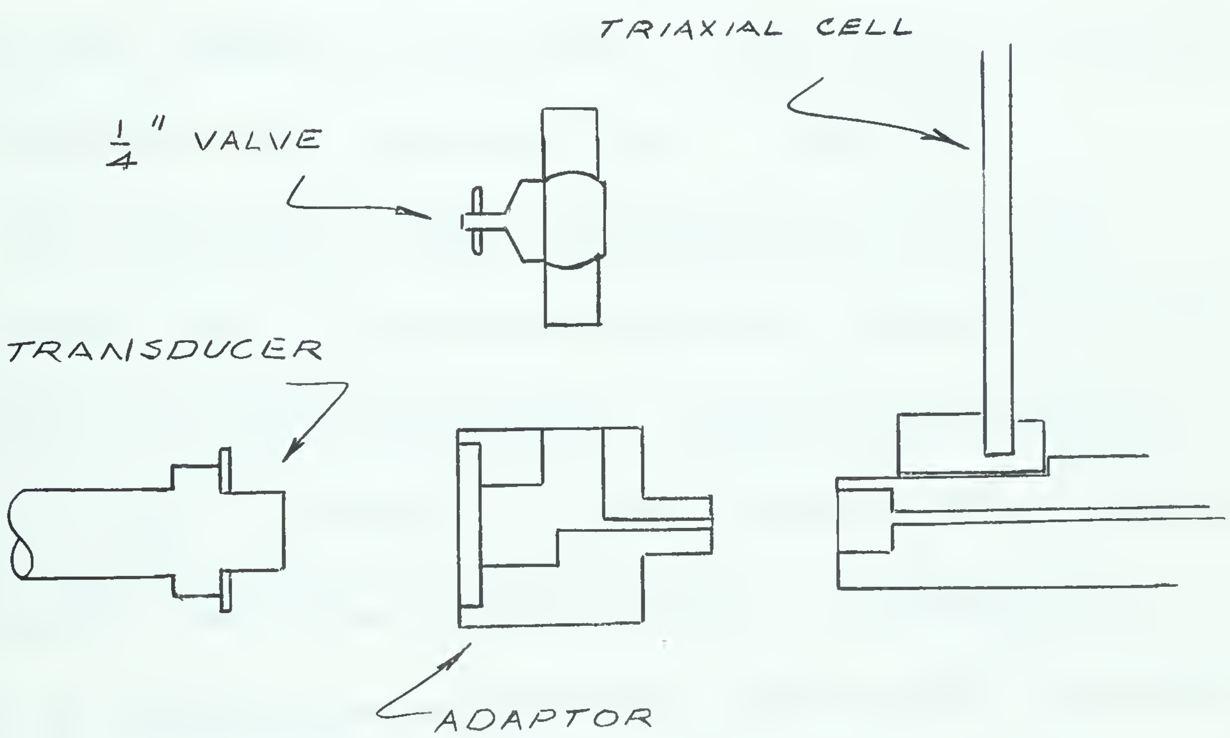


FIGURE B.2 Transducer Adaptor

Transducer signals may be recorded in several ways. Since the out-put signal is expressed in millivolts per volt perhaps the simplest system is a vacuum tube millivolt meter with a common six volt dry cell battery as the power supply. One voltmeter can monitor an entire bank of transducers and if the readings are taken by a technician the switching can be done by hand. Such systems have been found very satisfactory (Rowe and Bardon, 1964). SR4 strain indicators may be used in much the same way. If automatic recording is desirable, strip printers or X - Y plotters may prove useful. If a number of transducers must be served, a system involving a scanning unit capable of scanning a large number of transducers at suitable time intervals may be employed.

For purposes of this investigation a battery powered Baldwin Type L Strain Indicator was selected as the most suitable recording instrument. Since the transducer is affected by slight variations in power supply, it is necessary to periodically check the battery output. Although the transducer is temperature compensated the external circuitry, consisting of about five feet of five conductor cable, may be slightly influenced by variations in temperature.

By far the most important factor affecting accuracy is the "compliance" or "flexibility" of the system. The compliance of a base connected PT25 transducer is about 0.4 x 10^{-6} in.^{3}/psi (Richardson and Whitman, 1963) which compares very favourably with any other available pore pressure measuring system (Bishop and Henkel, 1962). This low value is only applicable if the system is completely deaired. In addition, accurate pore pressures are dependent on there being absolutely no leakage of water into or out of the system. For this reason the least possible number of intermediate valves is desirable.

B.4 Calibration

Each of the transducers was calibrated against a pre-checked Bourdon gauge. The transducers were adapted to the triaxial cells which were then filled with water. The transducer leads were connected to the posts of the strain indicator as shown in FIGURE B.3 .

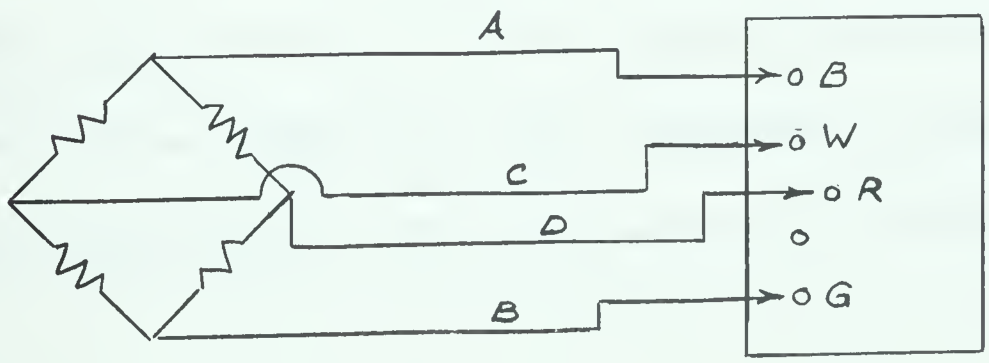


FIGURE B.3 Connections to Strain Indicator

Use of the strain indicator differs slightly from that normally associated with SR4 strain gauges. The transducer strain gauges have no specific gauge factor. Consequently the gauge factor dial on the indicator may be set at any convenient position so long as it remains unchanged during the test period. A change in the setting will alter the number of microinches per inch recorded for any given pressure increment.

The cell pressure was increased and lowered in increments and the corresponding readings in microinches per inch were recorded. The procedure was repeated several times with different gauge factor settings in an attempt to make the indicator record the applied pressure directly. This was found to be impossible. After several trials, the gauge factor dial was set at 2.15 and remained unaltered throughout the remainder of the program. The calibration curves for all three transducers are valid for this setting only.

Once the setting 2.15 was selected several readings were obtained and averaged for each increment of pressure. The resulting calibration curves are shown in FIGURE B.4 to

B.7 . They give the pressure on the diaphragm in terms of p.s.i. per division corresponding to any given number of divisions above that which represents atmospheric pressure. It should be noted that two curves are presented for the 100 p.s.i.a. transducer. FIGURE B.4 indicates the initial conditions of the instrument and is applicable to tests conducted prior to 12 December, 1964. Subsequent distortion necessitated recalibration. FIGURE B.5 applies to pore pressures measured after 12 December, 1964.

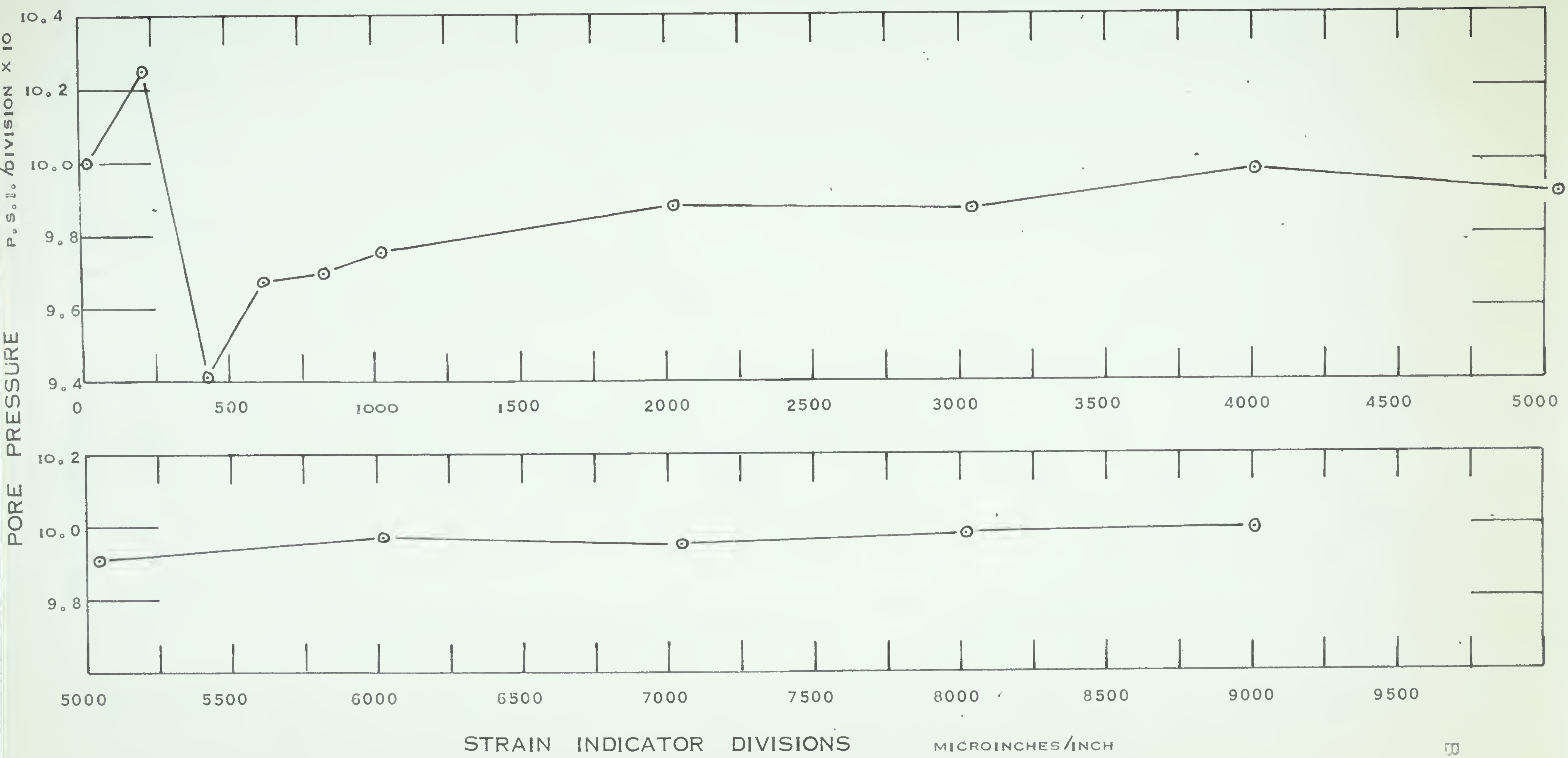


FIGURE B.4 CALIBRATION CURVE APT25-1C G.F. = 2.15

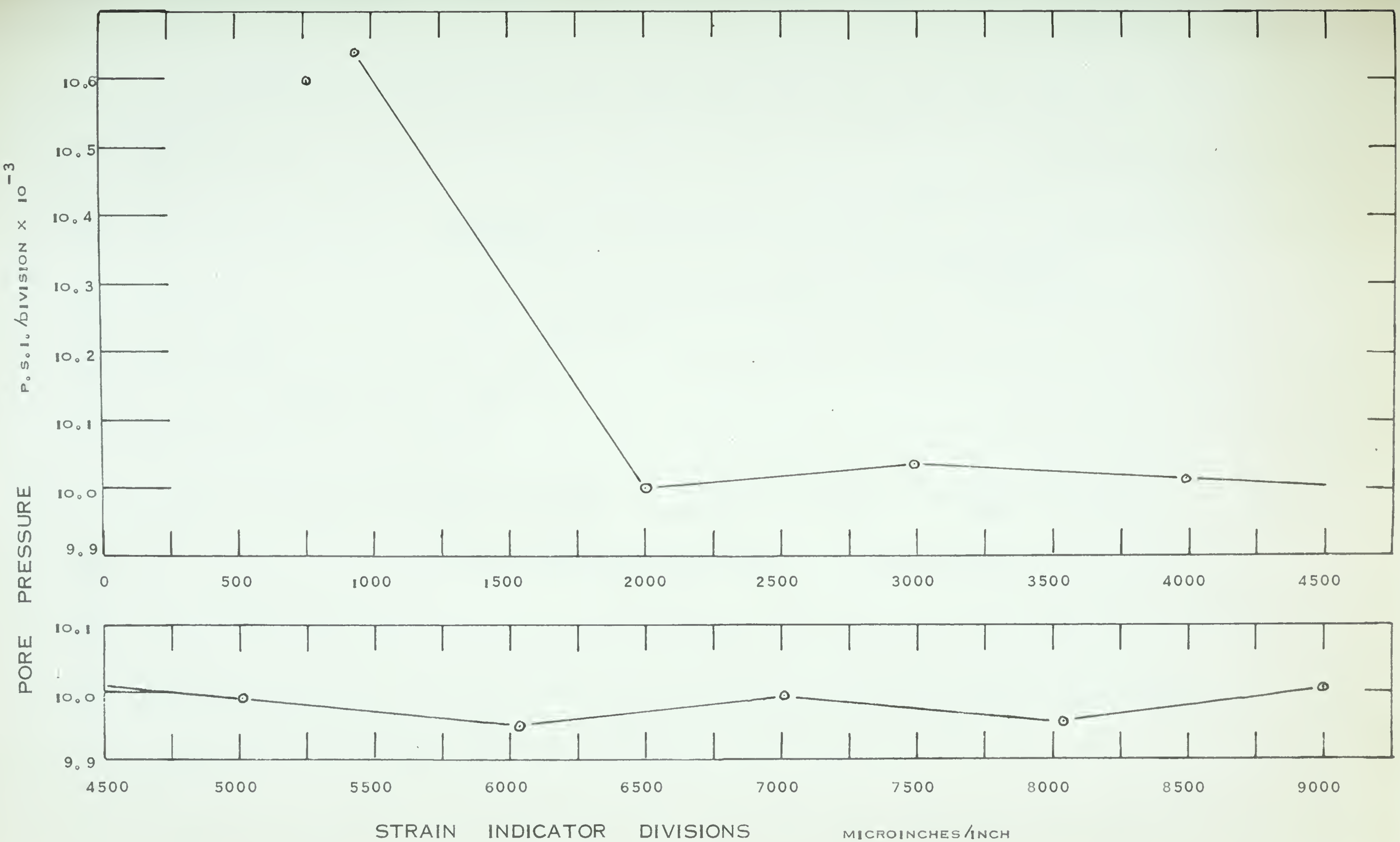


FIGURE B.5 CALIBRATION CURVE APT25-IC G.F. = 2.15

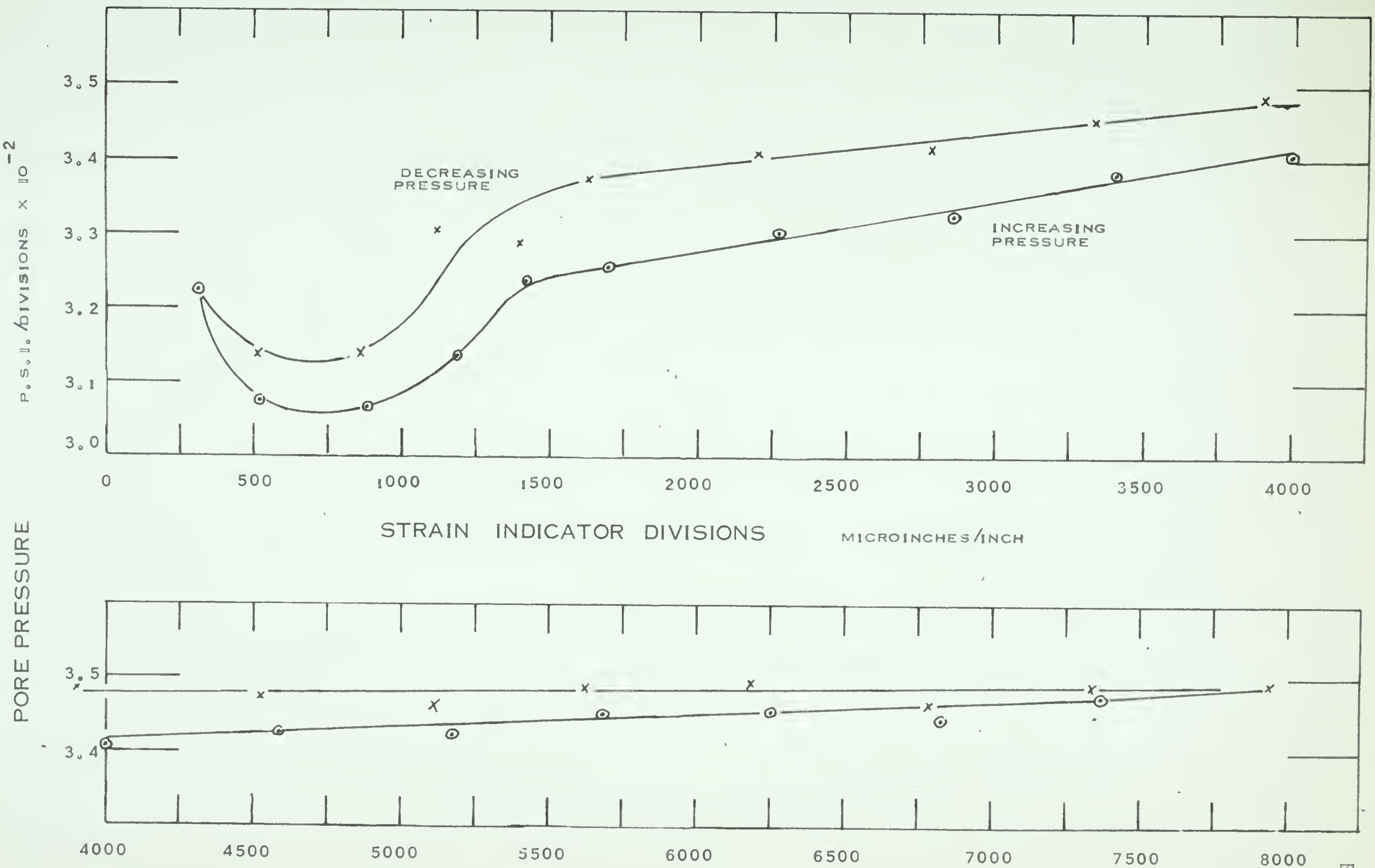


FIGURE B.6 CALIBRATION CURVE APT25 3C G.F. = 2.15

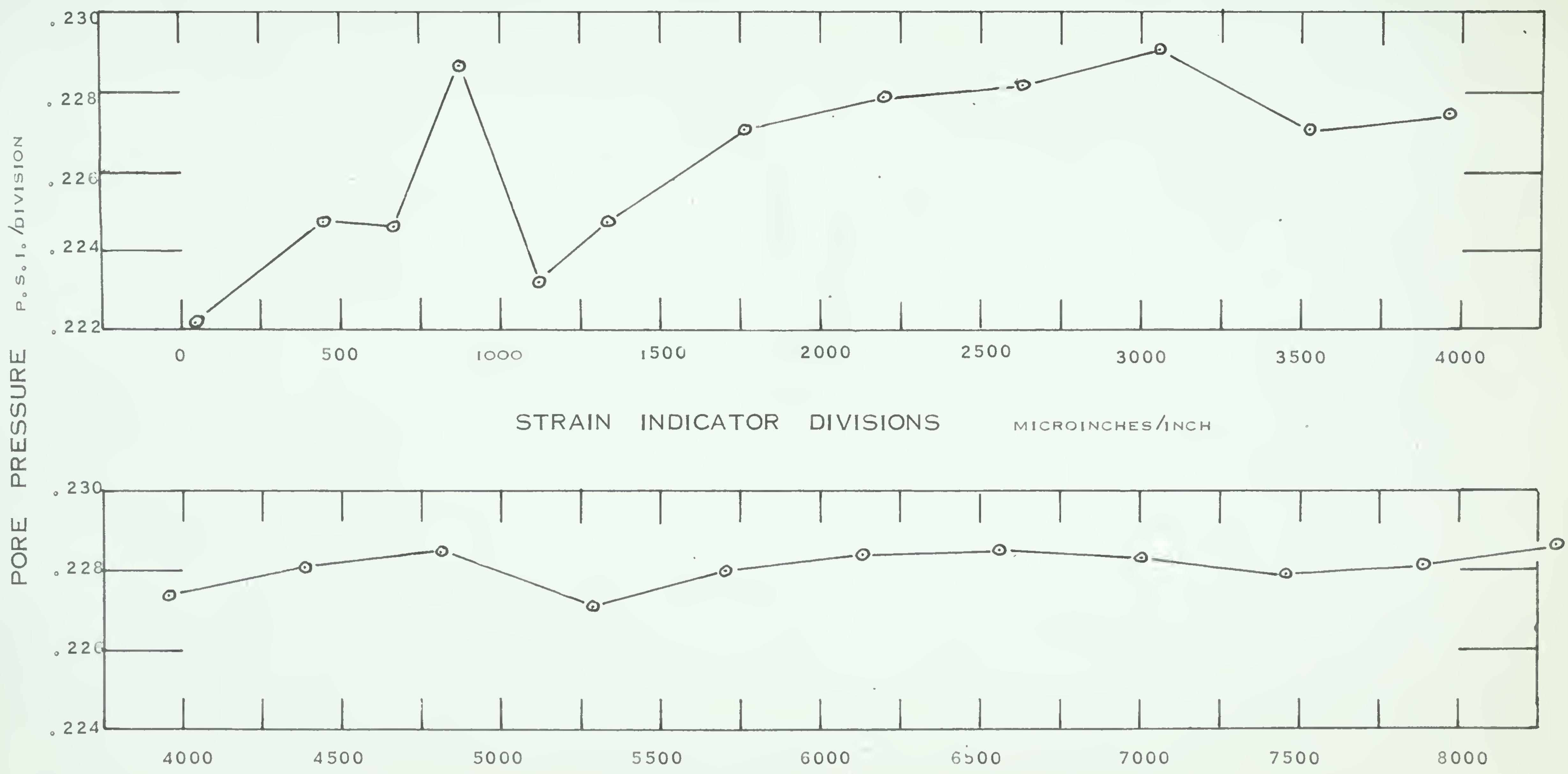
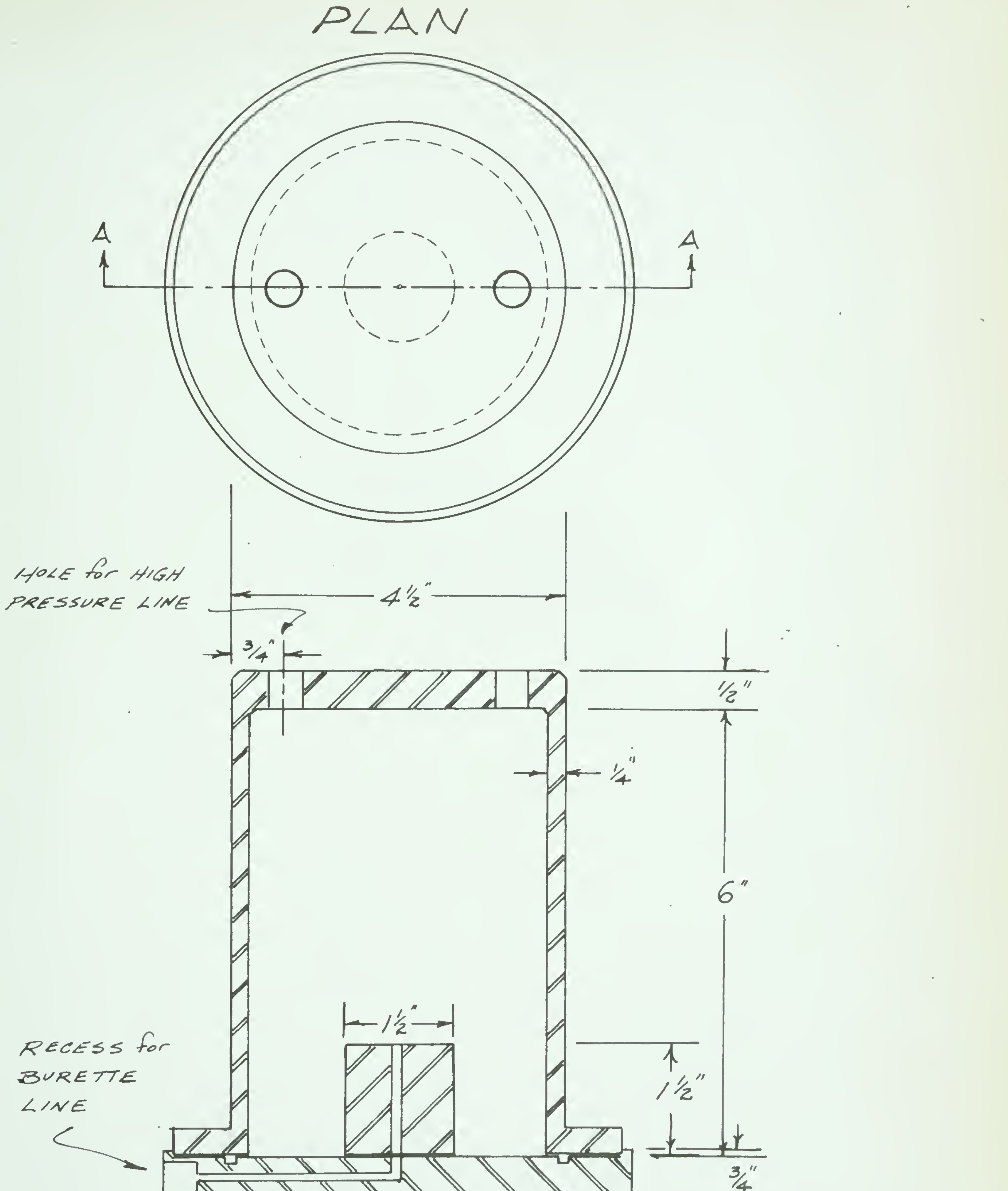


FIGURE B.7 CALIBRATION CURVE APT25-2M G.F. = 2.15

Appendix C

MEMBRANE TEST CELL



HIGH PRESSURE CELL

scale: approx. ½ size

GWH. 12 NOV 64

APPENDIX D
SASKATCHEWAN SILT

REPRESENTATIVE DATA

CONSOLIDATED – UNDRAINED

TESTS

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. —

Depth — sample S.S. 14

Engineer [illegible]

Technician —

Date of Set-up 31 JAN 65

Test Lateral Pressure 1203 psi

Sample Description Remolded Sask. Silt. very soft tears easily upon carving.

Initial Weight 194.45 gms

A12 - 66.138

initial atmos rdg 10865

	1	2	Aver	
Length, mm	81.38	81.22	81.30	Area Top A_T 11.879 sq.mm
Diam. mm Top	39.00	38.78	38.89	Area Centre A_C 11.977 sq.mm
Centre	39.10	39.00	39.05	Area Bottom A_B 11.897 sq.mm
Bottom	38.90	38.94	38.92	Average X-Sect Area

Original Volume 97.011 cc

$L_o = 3.201$

Average X-Sect Area $= \frac{1}{4}(A_T + 2A_C + A_B)$

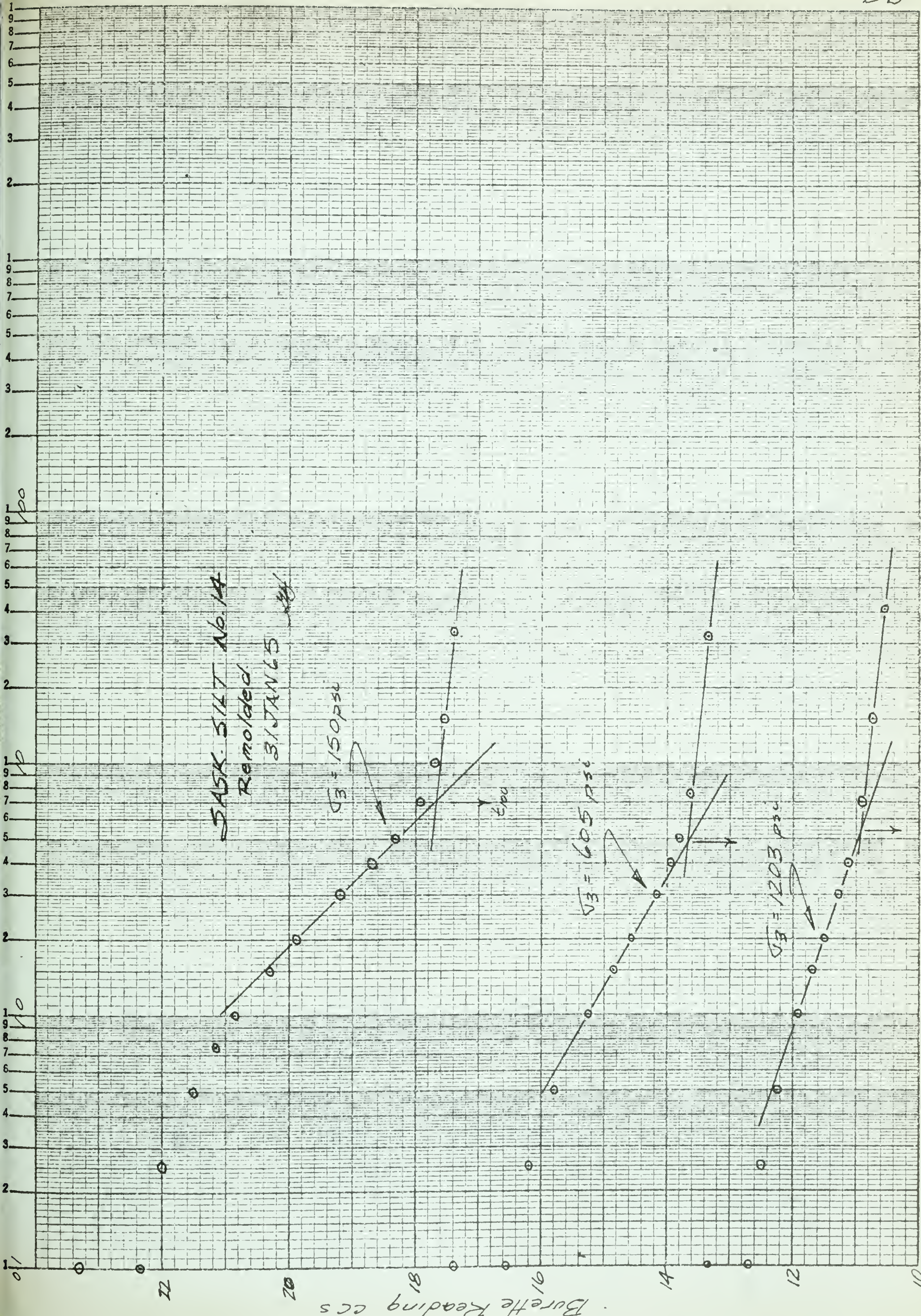
$= 11.933\ cm^2$

CONSOLIDATION DATA

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
1330	0	23.30	0
50 psi	6 secs	22.35	0.95
	15	22.00	1.30
	30	21.53	1.77
	45	21.18	2.12
	60	20.85	2.45
	90	20.32	2.98
	2 min	19.90	3.40
	3	19.20	4.10
	4	18.70	4.60
	5	18.33	4.97
	7	17.93	5.37
	10	17.70	5.60
	15	17.55	5.75
	33	17.40	5.90
1410	0	17.40	5.90
605 psi	6	16.55	6.75
	15	16.20	7.10
	30	15.80	7.50
	45		

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
σ_3 =	60 secs	15.26	8.04
605 psi	90	14.86	8.44
	2 min	14.58	8.72
	3	14.18	9.12
	4	13.96	9.34
	5	13.81	9.49
	7.5	13.65	9.65
	32 min.	13.35	9.95
31 1445	0	13.35	9.95
1203 psi	6 secs	12.70	10.60
	15	12.50	10.80
	30	12.25	11.05
	60	11.92	11.38
	90	11.70	11.60
	2 min	11.51	11.79
	3	11.30	12.00
	4	11.14	12.16
	7	10.92	12.38
	15	10.74	12.56
	41	10.55	12.75
	45	10.53	


SASK. SILT No. 1A
Remolded
31 JAN 65
σ3 = 150 psi
σ3 = 605 psi
σ3 = 1203 psi
t100
Time, mins.
0.1
1.0
10
100
Burette Reading cc's
22
20
18
16
14
12
10

D3

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING

SOIL MECHANICS LABORATORY
TRIAXIAL COMPRESSION TEST
PORE PRESSURE REACTION TEST

Sample No 55 14

Sample Desc Remolded.

Kg/sq. cm on cell 1203 psi

Initial Rdg - 10860

back pres for 23 hrs

"LOAD" 50psi		50psi	
t min	Pp Kg/cm^2		
0	10860	11080	49.15 ~~71.12~~
0.1	10870	120	58.14
0.25		150	64.90
0.50		170	69.38
1.0	10872	180	71.68
2.	10875	185	72.80
3			
4			47%
5			
bp	0psi	50psi	

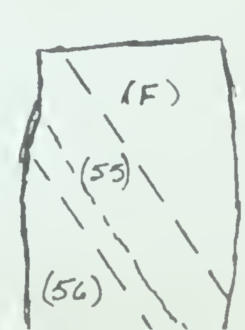
"UNLOAD" 50psi		50psi	
t min	Pp Kg/cm^2		
0	10880	11185	72.8
0.1	875	140	62.64
0.25		115	56.99
0.50		11104	
1.0		11100	53.62
2.			
3			
4			
5			
	0psi	50psi	

Final Moisture Content	(F) T	(55) C	(56) B	Initial m.c.
Wet Wt. plus tare	65.51	77.63	122.60	66.138
Dry Wt. plus tare	61.00	71.51	110.11	56.716
Wt. of water	4.51	6.12	12.49	9.422
Wt. and No. of tare	32.30	33.86	34.45	A12 - 18.946
Wt. of dry soil	28.70	37.65	75.66	37.770
Final M.C.	15.71%	16.25%	16.51%	24.95%

Final Volume

Wt. Hg + Tare	1313.60	
Tare	170.10	
Wt. Hg	1143.50	
Temp.	24.9 °c	13.5344
Vol. Hg	84.49cc	

Total final wt = 162.30gms



very faint suggestion of shear plane

D4
Sheet 1

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

XIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. —

Depth — Sample SASK SILT 1A

Engineer [signature]

Technician —

Date of Test 31 JAN 65

Test Lateral Pressure σ_3 1203 psi

Back Pressure 50 psi

Remarks

at Failure

$-\sigma_3) =$ 886.5 psi $\sigma_1 =$ 2089.5 psi.

871.6 psi

1217.9 psi

331.4 psi

n 12 % %

Area Correction Factor $K = 1.125$

$L_o = 3.201''$ PR # 867 nominal rate of strain = 0.005 in/min

me n	Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$		Pore Press kg/cm² P_p	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_p}{\sigma_1 - \sigma_3}$
05	0	11.933	240	1.929	0	11 135	61.5	1191.5	1191.5	1.00	0	∞
	1.6	11.939	268	1.927	71.7	197	75.5	1249.2	1177.5	1.060	.05	0.355
	3.2	11.945	283	1.926	110.0	250	87.5	1275.5	1165.5	1.094	.10	0.34
09	6.4	11.957	312	1.925	184.2	400	121.3	1315.9	1131.7	1.162	.20	0.39
12	12.8	11.981	379	1.920	353.5	890	231.8	1374.7	1021.2	1.346	.40	0.51
14	19.2	12.005	436	1.917	497.0	12 450	359.3	1390.7	893.7	1.556	.60	0.62
17	32.0	12.054	497.5	1.913	648.8	13 250	544.9	1356.9	708.1	1.916	1.00	0.76
21	48	12.115	532	1.911	731.3	13 908	698.0	1286.3	555	2.317	1.5	0.89
24	64	12.177	545	1.910	759.5	14 336	789.7	1222.8	463.3	2.64	2.0	0.97
30	96	12.302	554	1.910	774.1				~~335.8~~		3	
36	128	12.430	561.8	1.910	785.3	890	917.2	1121.1	335.8	3.34	4	1.10
	160	12.561	571	1.909	798.8	955	932.0	1119.8	321.0	3.49	5	1.10
	192	12.694	582.2	1.908	816.7	990	940.0	1129.7	313.0	3.61	6	1.09
54	224	12.831	593.2	1.907	833.3	991	940.2	1146.1	312.8	3.66	7	1.07
	256	12.970	604.3	1.907	850.5	990	940.0	1163.5	313.0	3.72	8	1.05
06	288	13.113	615	1.905	864.7	980	937.7	1180	315.3	3.74	9	1.03
12	320	13.259	624.2	1.905	876.2	960	933.2	1196	319.8	3.74	10	1.01
18	352.1	13.408	632	1.905	884.3	940	928.6	1208.7	324.4	3.725	11	0.99
24	384.1	13.560	637.4	1.905	886.5	910	921.6	1217.9	331.4	3.675	12	0.98
30	416.1	13.716	640.5	1.905	883.3	890	916.8	1219.5	336.2	3.63	13	0.98
36	448.1	13.876	634.2	1.905	859.3	885	915.7	1196.6	337.3	3.55	14	1.01

05

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING

SOIL MECHANICS LABORATORY
TRIAXIAL COMPRESSION TEST COMPUTATIONS

Sample No. SASK SILT No 14
Sample Desc. Remolded

BEGINNING OF TEST

Original Vol of Specimen 97.011 cc

Wt. of Soil Solids in Specimen $\frac{194.45}{124.95}$ = 155.62

Vol. of Soil Solids $G_s = 2.67$ 58.285

Vol of Voids 38.726

Original Void Ratio, e_o 0.664

Original Porosity, n, 39.90 %

Wt. of Water 38.83 gms

Original Degree of Saturation, S, 100.26 %

Original Wet Wt., lb/cu. ft. 125.07

Original Dry Wt., lb/cu. ft. 100.10

END OF TEST

Final Vol of Specimen (by Hg immersion) 84.49 cc

Vol. of Soil Solids 58.29

Vol. of Voids 26.20 cc

Final Void Ratio, e_f 0.449

Final Porosity, n, 30.99 %

Wt. of Water 182.30 - 155.62 = 26.68

Final Degree of Saturation 101.8 %

Final Wet Wt. lb/cu.ft. 134.63 pcf

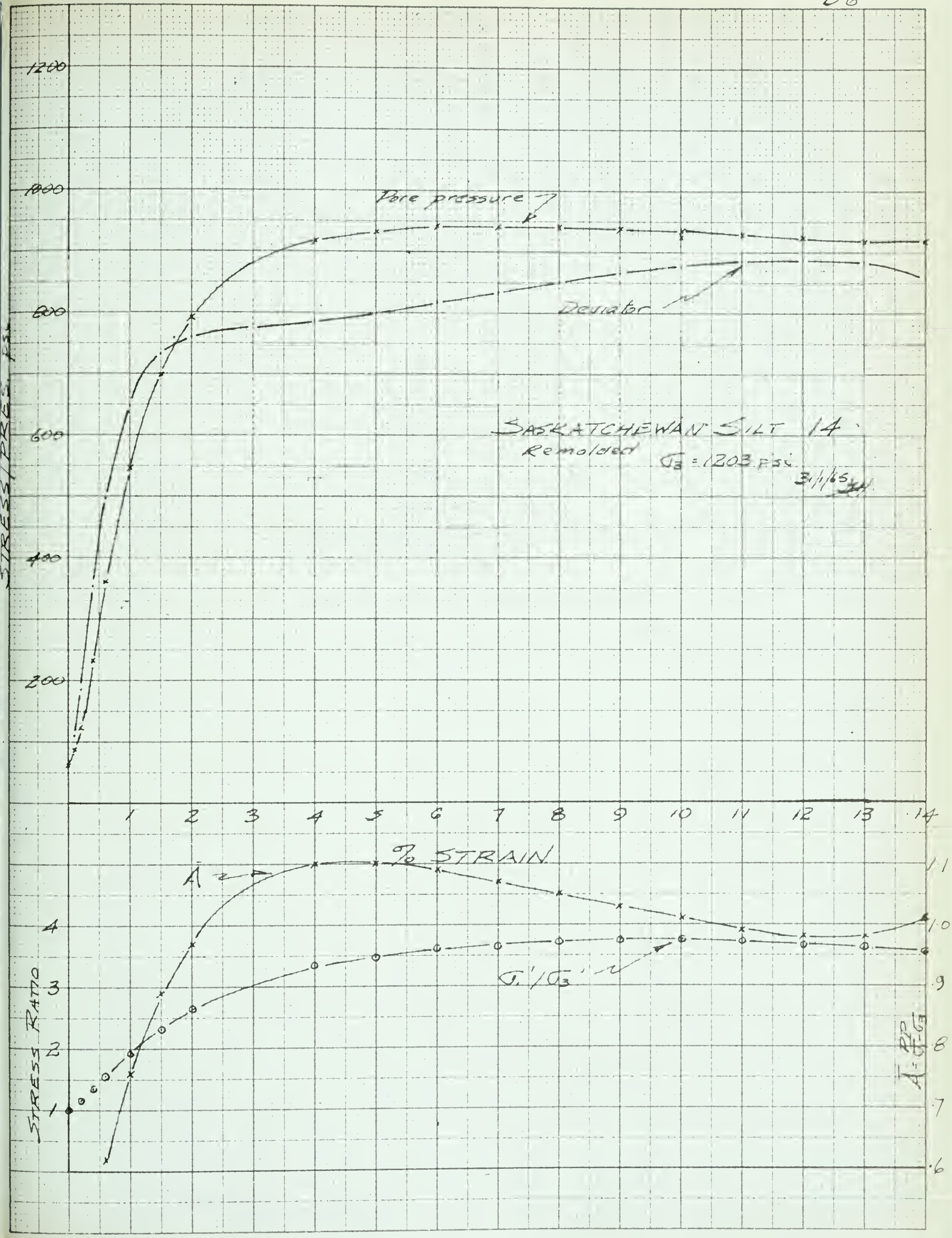
Final Dry Wt. lb/cu.ft. 114.93

Wet wt. of specimen at beginning of test 194.45 gm.

Wet wt. of specimen at end of test 182.30 gm.

Weight loss 12.15 gm.

Vol. change from burette rdg. discrepancy 0.62 cc.



1200
1000
800
600
400
200
STRESS/PRES. PSI
Pore pressure
Deviator
SASKATCHEWAN SILT 14
Remolded
σ_3 = 1203 psi.
3/1/65
1 2 3 4 5 6 7 8 9 10 11 12 13 14
% STRAIN
A
σ_1'/σ_3'
STRESS RATIO
1 2 3 4
1.1
1.0
.9
.8
.7
.6
$A = \frac{PP}{\sigma_1 - \sigma_3}$

APPENDIX E

BEARPAW SHALE

REPRESENTATIVE STRENGTH DATA

AND

CONSOLIDATION CHARACTERISTICS

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

E1

RIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project: THESIS

Hole No.: SSRD 3754

Depth: 103 ft sample: BPS 11

Engineer: [illegible]

Technician: —

Date of Set-up: 19 MARCH 65

Test Lateral Pressure: 120 psi

ample Description: UNDISTURBED Bearpaw Shale. stiff grey clay shale w/ sandstone lens - one small shell fossil.

nitial Weight: 172.80 gms

	1.	2.	Aver:	
ngth, mm	81.32	81.14	81.23	
m. mm Top	36.74	36.78	36.76	Area Top A_T 10.613 sq.mm
Centre	36.90	36.58	36.74	Area Centre A_C 10.602 sq.mm
Bottom	37.10	36.24	36.67	Area Bottom A_B 10.561 sq.mm

ginal Volume: 86.063 cc

$L_o = 3.198''$

Average X-Sect Area $= \frac{1}{4}(A_T + 2A_C + A_B)$

$= 10.595$

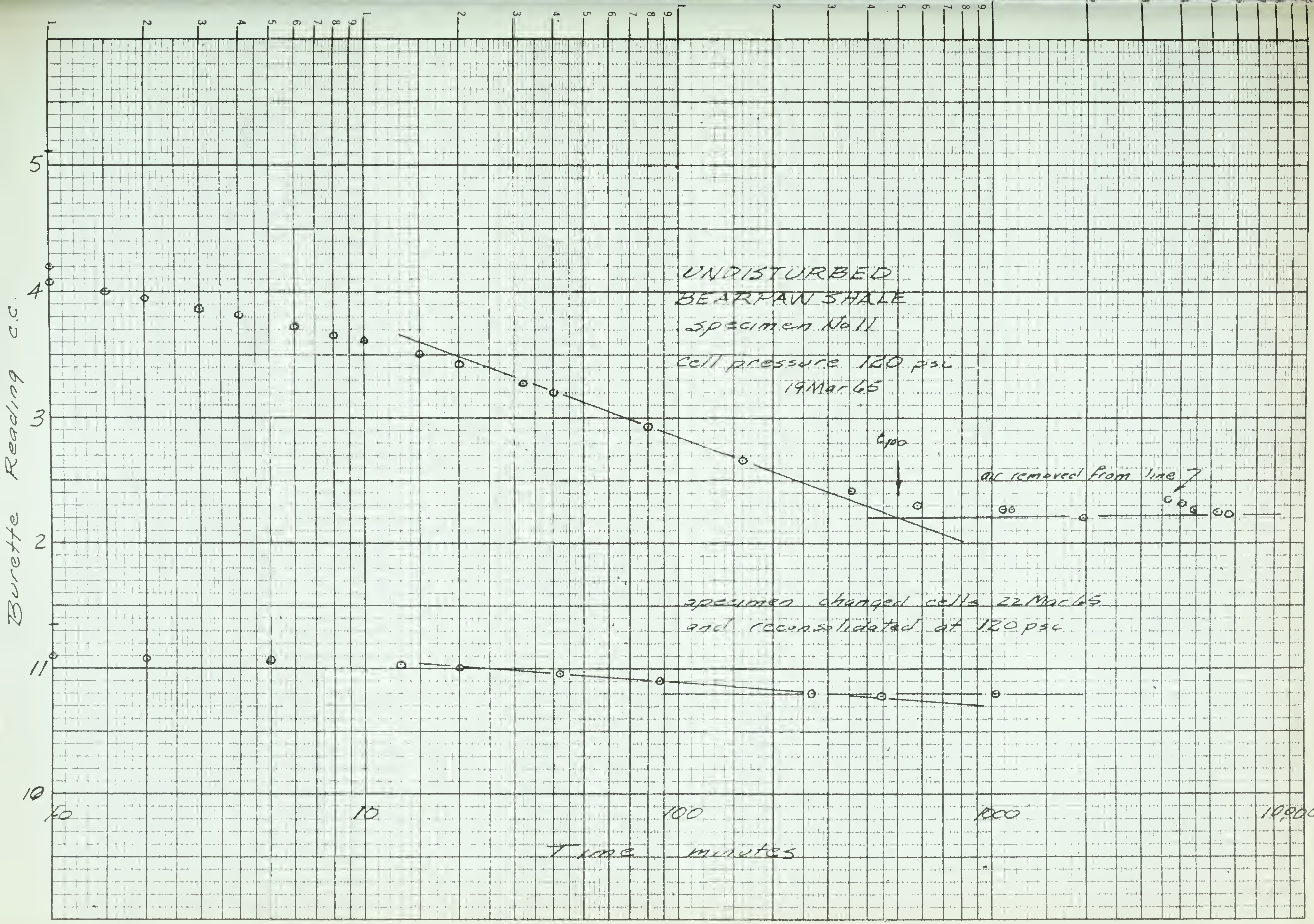
NSOLIDATION DATA

* changed cells

te me	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
345	0	5.11	0
	6 secs	-	-
	15	4.20	0.91
	30	4.15	0.96
	45	4.11	1.00
	1 min	4.07	1.04
	1.5	4.00	1.11
	2	3.95	1.16
	3	3.87	1.24
	4	3.81	1.30
	6	3.73	1.38
	8	3.66	1.45
	10	3.61	1.50
	15	3.50	1.61
	20	3.42	1.69
	32	3.28	1.83
	40	3.20	1.91
	80	2.93	2.18
	160	2.66	2.45
940	355	2.41	2.70

CONSOLIDATION DATA – cont'd

Date Time	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
192325	580	2.30	2.81
200755	1090	2.27	2.84
200900	1155	2.26	2.85
202230	1965	2.20	2.91
211000	3655	2.35	2.76
1620	4035	2.32	2.79
2300	4435	2.27	2.84
221010	5105	2.25	2.86
* 1420	5355	2.22	2.89
221520	0	11.35	—
	6 secs	11.10	
	15	11.10	
	30	11.10	2.89
	60	11.10	
	2 mins	11.08	2.91
	5	11.06	2.93
	13	11.02	2.97
	20	11.00	2.99
	42	10.96	3.08
1647	87	10.90	3.09
1947	267	10.80	3.19
2245	445	10.79	3.20
230825	1025	10.80	3.19


UNDISTURBED
BEARPAW SHALE
specimen No 11
Cell pressure 120 psi
19 Mar 65
t_{po}
air removed from line
specimen changed cells 22 Mar 65
and reconsolidated at 120 psi
Burette Reading c.c.
Time minutes
1
10
100
1000
10,000
5
4
3
2
11
10

E3

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING

SOIL MECHANICS LABORATORY
TRIAXIAL COMPRESSION TEST
PORE PRESSURE REACTION TEST

Sample No Bearpaw 11
Sample Desc. UNDISTURBED
Kg/sq. cm on cell 120 psi

Inital atmos rdg = 11400
20 psi backpres applied 230830

"LOAD" 8 psi @ 251900 hrs

t min	Pp Kg/cm^2	
0	11950	16.89 psi
0.1	965	
0.25	970	
0.50	970	
1.0	975	13 1/0
2.	980	
3		
4	980	
5	985	17.93 psi

bp 20 psi

"UNLOAD" 8 psi

t min	Pp Kg/cm^2	
0	11985	17.93 psi
0.1	980	
0.25	979	
0.50	979	
1.0	978	
2.	975	
3	970	
4	970	17.50 psi
5		

20 psi

Final Moisture Content

	Top	Centre	Bottom		
Wet Wt. plus tare	69.60	81.81	104.62		66.10
Dry Wt. plus tare	62.76	72.77	90.68		58.12
Wt. of water	6.84	9.04	13.94		7.98
Wt. and No. of tare	(55) 33.39	(56) 34.47	(61) 32.60	47LT	24.22
Wt. of dry soil	29.37	38.30	58.08		33.90
Final M.C.	23.28%	23.60	24.00%		23.54%

Final Total weight = 173.60 gms

Final Volume

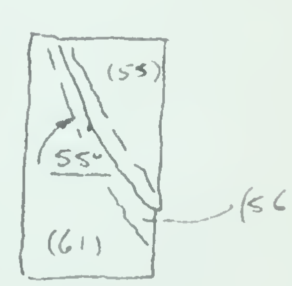
Wt. Hg + Tare 1325.80

Tare 170.95

Wt. Hg 1154.85

Temp. 26°C 13.5316

Vol. Hg 85.345 cc



UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

IAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. 55RD3754

Depth 103 ft Sample BPS No 11

Engineer [illegible]

Technician —

Date of Test 25 MAR 65

Test Lateral Pressure σ_3 120 psi

Back Pressure 20 psi

Remarks APT 25-3C atmos = 11400
approx rate of strain = 1.3×10^{-5} in/min
PR #783

Area Correction Factor $K = 1.025$

ta at Failure

$(\sigma_1 - \sigma_3) =$ 245 psi $\sigma_1 =$ ~~40~~

= 37 psi

= 328 psi

= 83 psi

ain 1.90 %

$L_o = 3.198''$

ime min	Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$		Pore Press kg/cm² P_P	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\bar{\sigma}_1 / \bar{\sigma}_3$	Axial Comp. Strain %	$\bar{A} = \frac{P_P}{\sigma_1 - \sigma_3}$
51940	500	10.595	50	.4487	0	11 980	17.81	122.19	122.19	1.00	0	—
2230	501	10.598	61	4479	6.71	12 138	22.58	124.13	117.42	1.06	.031	.38
0830	505.2	10.612	109	4400	35.05	477	33.41	141.64	106.59	1.33	.163	.383
1200	507	10.618	126.7	4335	44.59	610	38.05	146.54	101.95	1.44	.219	.405
1330	507.6	10.620	132	4325	47.56	660	39.88	147.88	100.12	1.48	.238	.418
1600	508.2	10.622	138.5	4320	51.32	696	41.21	150.61	98.79	1.52	.256	.413
2100	508.9	10.625	141.4	4310	52.82	700	41.38	151.44	98.62	1.54	.278	.405
0915	512	10.635	174.5	4260	71.14	820	45.82	165.32	94.18	1.76	.375	.363
1420	514.2	10.642	199.3	4239	84.98	930	49.65	175.33	90.35	1.94	.444	.349
2315	518.4	10.656	238.9	4226	107.41	13 092	55.07	192.34	84.93	2.26	.575	.327
0930	523.4	10.673	281.6	4240	132.43	250	60.46	211.97	79.54	2.66	.732	.306
1400	525.8	10.681	300.0	4248	143.31	305	62.29	221.02	77.71	2.84	.807	.295
2140	529.2	10.693	328.1	4259	159.92	360	64.19	235.73	75.81	3.11	.913	.276
0830	534.6	10.711	366.1	4269	182.17	440	66.91	255.26	73.09	3.49	1.082	.258
1100	536.1	10.716	375.5	4271	187.66	507	69.21	258.45	70.79	3.65	1.129	.262
1535	538.5	10.724	389.7	4273	195.86	510	69.31	266.55	70.69	3.77	1.204	.252
1840	540.1	10.730	399.2	4275	201.37	505	69.15	272.22	70.85	3.84	1.254	.244
2150	541.9	10.736	408.4	4275	206.60	490	68.66	277.94	71.34	3.90	1.310	.236
0845	547.8	10.756	443.1	4279	226.57	410	65.93	300.64	74.07	4.06	1.495	.203
1320	551	10.767	454.8	4280	233.17	420	66.26	310.91	73.74	4.22	1.595	.198
1650	553.2	10.774	462.5	4280	237.50	315	64.70	312.80	75.30	4.15	1.664	.188
1920	555	10.780	468.2	4280	240.67	350	63.86	316.81	76.14	4.16	1.720	.178
0845	582.1	10.693	331.9	4260	162.13	12 610	38.12	264.05	101.88	2.59	2.567	.112
1000	584	10.693	328.8	4258	160.88	618	38.37	261.91	101.63	2.58	2.627	.115
1450	590	10.640	316.2	4255	158.59	620	38.43	255.16	101.57	2.51	2.814	.120
1025	595.9	10.588	309.6	4252	150.34	598	37.62	252.72	102.38	2.47	2.999	.117
0810	606	10.537	310.7	4253	161.77	460	32.86	258.91	107.14	2.42	3.315	.085
1425	612.9	10.485	301.2	4249	146.73	510	34.63	252.10	105.37	2.39	3.530	.100
1530	614	10.485	300.9	4249	146.55	510	34.63	251.92	105.37	2.39	3.565	.100

E5

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING

SOIL MECHANICS LABORATORY
TRIAXIAL COMPRESSION TEST COMPUTATIONS

Sample No. BPS 11
Sample Desc. UNDISTURBED

BEGINNING OF TEST

Original Vol of Specimen 86.063 cc

Wt. of Soil Solids in Specimen $\frac{172.80}{1.2354}$ = 139.87

Vol. of Soil Solids ($G_s = 2.70$) 51.803

Vol of Voids 34.260

Original Void Ratio, e_o 0.661

Original Porosity, n, 39.80 %

Wt. of Water 32.93

Original Degree of Saturation, S, 96.12 %

Original Wet Wt., lb/cu. ft. 125 pcf

Original Dry Wt., lb/cu. ft. 101.2 pcf

END OF TEST

Final Vol of Specimen (by Hg immersion) 85.345

Vol. of Soil Solids 51.803

Vol. of Voids 33.542

Final Void Ratio, e_f 0.647

Final Porosity, n, 39.28 %

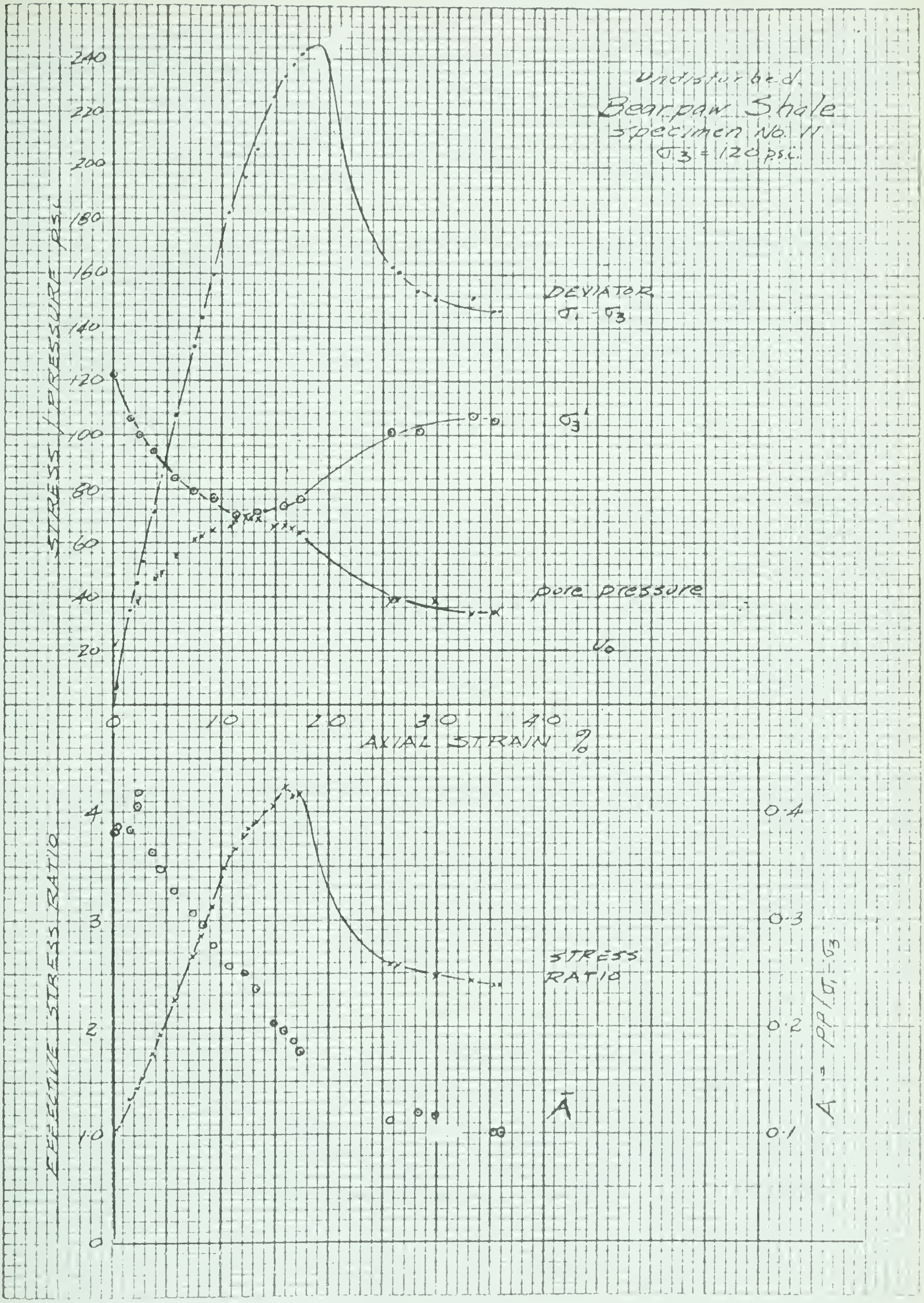
Wt. of Water 173.60 - 139.87 = 33.73

Final Degree of Saturation 100.57 %

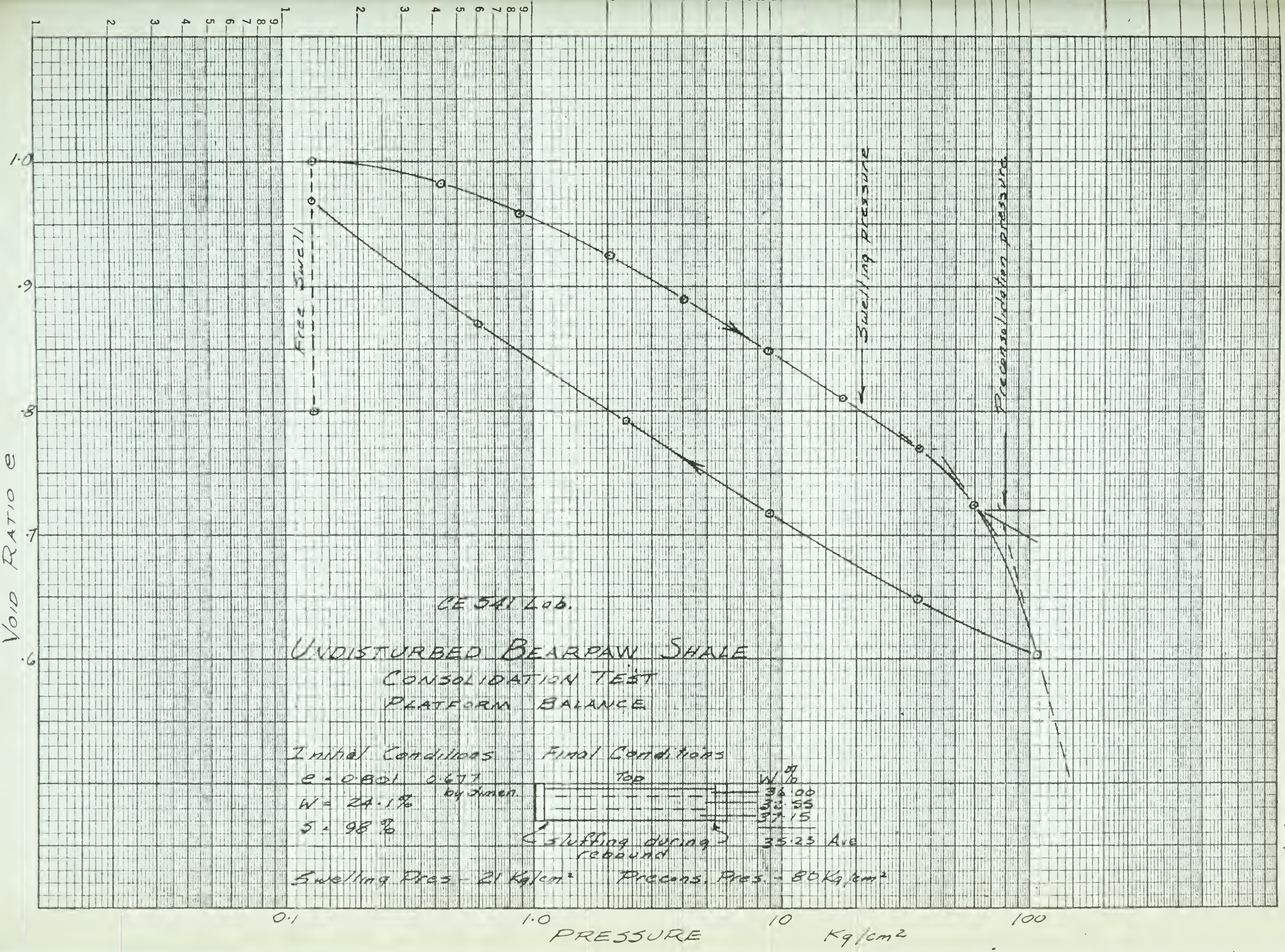
Final Wet Wt. lb/cu.ft. 126.2 pcf

Final Dry Wt. lb/cu.ft. 102 pcf

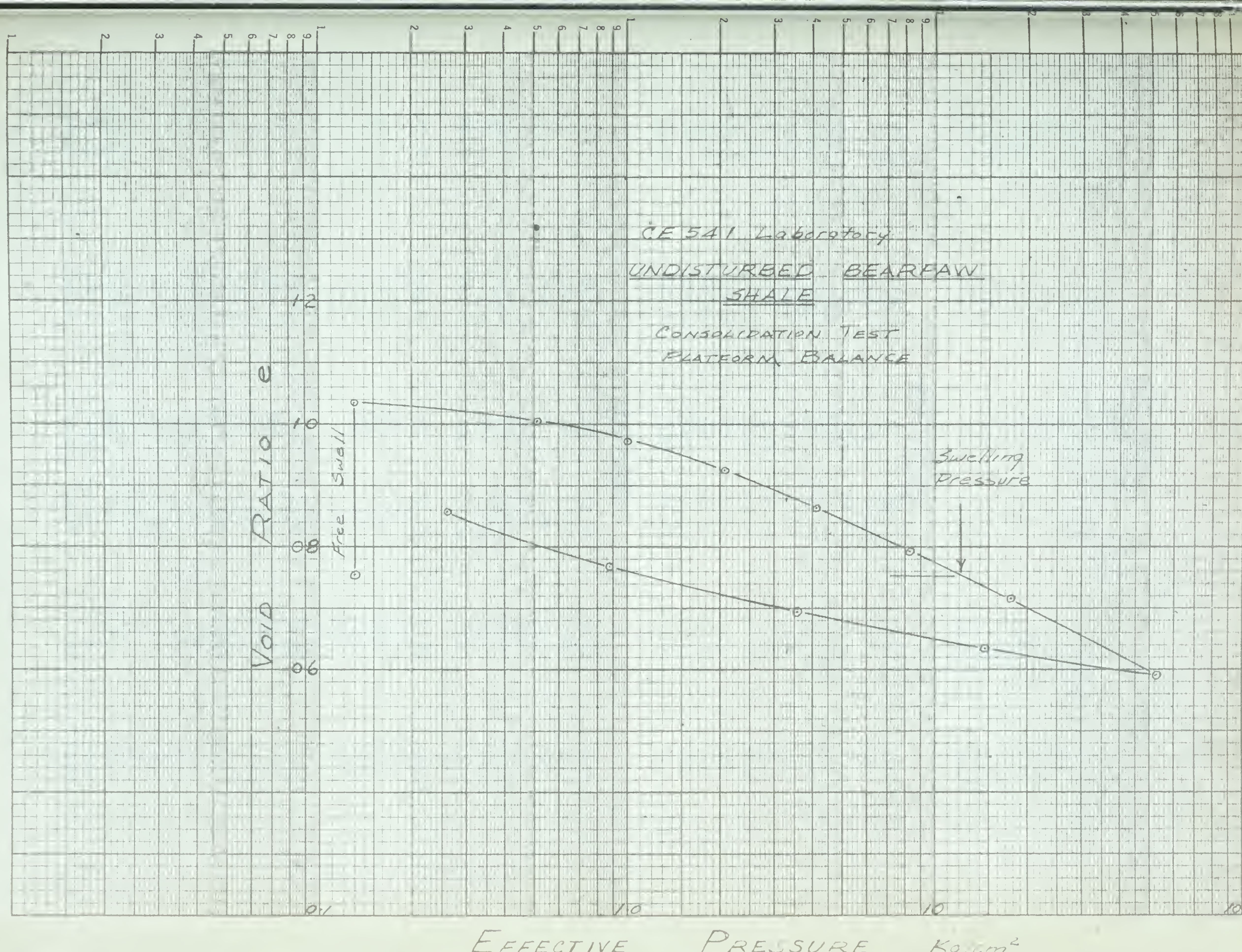
			vols	
Wet wt. of specimen at beginning of test	172.80	gm.	86.063	cc
Wet wt. of specimen at end of test	173.60	gm.	85.345	cc
Weight loss	- 0.80 gm	gm.	+ .718	cc
Vol. change from burette rdg. discrepancy	4 cc.	cc.	2.47	


Undisturbed
Bearpaw Shale
specimen No. 11
$\sigma_3 = 120$ psi.
STRESS / PRESSURE psi.
240
220
200
180
160
140
120
100
80
60
40
20
0
DEVIATOR
$\sigma_1 - \sigma_3$
σ_3'
pore pressure
u_0
0
1.0
2.0
3.0
4.0
AXIAL STRAIN %
EFFECTIVE STRESS RATIO
4
3
2
1.0
0
STRESS
RATIO
$\bar{A}$
0.4
0.3
0.2
0.1
$A = pp / \sigma_1 - \sigma_3$

E7


CE 541 Lab.
UNDISTURBED BEARPAW SHALE
CONSOLIDATION TEST
PLATFORM BALANCE
Initial Conditions
e = 0.801 0.677 by Simen.
W = 24.1%
S = 98%
Final Conditions
Top
W %
36.00
32.55
37.15
35.23 Ave
sluffing during rebound
Swelling Pres - 21 Kg/cm²
Precons. Pres. - 80 Kg/cm²
Free Swell
Swelling pressure
Preconsolidation pressure
VOID RATIO e
PRESSURE Kg/cm²
1.0
.9
.8
.7
.6
0.1
1.0
10
100

E7a


CE 541 Laboratory
UNDISTURBED BEARPAW
SHALE
CONSOLIDATION TEST
PLATFORM BALANCE
Swelling
Pressure
Free Swell
VOID RATIO e
1.2
1.0
0.8
0.6
0.1
1.0
10
100
EFFECTIVE PRESSURE Kg/cm²

E8

TRIAXIAL CONSOLIDATION
DATA Rec'd from
PFRA Soil Mechs Branch
(Rivard, 26 Nov 64)

SOFT SHALE

Water Content %	lb/cu ft γ wet	psi σc	Volume Change Butette-cc	Volume Change Weight-gms	t 100-min
30.7	122.6	70	-1.49	-1.00	900
38.0	115.8	100	-5.23	-5.48	900
37.8	115.2	25	+.04	-.05	200
37.8	115.5	50	-1.97	-1.95	2200
38.0	115.4	100	-5.34	-4.85	1800
36.2	117.0	150	-5.72	-5.62	1900

MEDIUM SHALE

29.2	118.0	10	+1.67	+2.41	150*
29.8	118.0	50	-1.43	-.48	120
29.5	118.0	100	-4.14	-3.43	240
29.1	119.5	150	-3.72	-3.06	700
29.4	120.8	10	+1.39	+2.43	400*
30.0	119.6	50	-.17	-.11	400
29.5	122	100	-1.30	-.58	440
29.6	120.9	150	-2.05	-1.77	1500
25.2	126.1	25	-.37	-.40	200
25.9	123.4	50	-2.59	-2.17	150
28.1	123.3	100	-5.30	-5.08	250
26.9	124.0	150	-5.51	-5.73	100

HARD SHALE

26.1	124.9	10	+3.09	+3.19	150*
26.2	125.0	45	+.66	+.98	350*
26.7	124.0	90	-.26	+.17	800
25.6	124.0	90	-.16	+.17	1000
25.9	125.4	90	-.12	+.37	800
26.0	124.4	90	+.015	+.48	700*
26.7	124.0	150	-1.20	-.75	1200
25.8	124.2	10	+2.73	+2.83	1000*
26.6	124.4	45	+.48	+.90	900*
26.8	125.0	90	-.19	+.50	800
25.9	124.8	90	-.17	+.36	600
26.3	124.4	150	-1.25	-.94	1500
26.2	127.0	10	+2.19	+2.48	1800*
27.4	124.2	30	+.79	+1.33	650*
26.7	217.0	50	+.47	+.79	1200*
26.3	126.5	60	+.01	+.48	1400*
28.3	126.3	80	-.19	-.09	200
26.5	127.0	100	-.30	-.20	800
28.0	126.4	120	-.67	-.38	2500
25.8	126.6	150	-1.58	-.56	600
25.4	126.5	25	+.83	+.80	600*
26.5	126.2	50	+.53	+.59	600*
26.3	126.2	100	-.93	-.77	1800
26.9	125.1	150	-1.99	-1.80	3000

* Swelling